INTERNATIONAL LIBRARY OF TECHNOLOGY

A SERIES OF TEXTBOOKS FOR PERSONS ENGAGED IN THE ENGINEERING PROFESSIONS AND TRADES OR FOR THOSE WHO DESIRE INFORMATION CONCERNING THEM. FULLY ILLUSTRATED AND CONTAINING NUMEROUS PRACTICAL EXAMPLES AND THEIR SOLUTIONS

WATER SUPPLY
SEWERAGE
PURIFICATION OF WATER
SEWAGE PURIFICATION AND DISPOSAL
IRRIGATION

SCRANTON:
INTERNATIONAL TEXTBOOK COMPANY

98

PRINTED IN THE UNITED STATES

10240

98

PREFACE

The International Library of Technology is the outgrowth of a large and increasing demand that has arisen for the Reference Libraries of the International Correspondence Schools on the part of those who are not students of the Schools. As the volumes composing this Library are all printed from the same plates used in printing the Reference Libraries above mentioned, a few words are necessary regarding the scope and purpose of the instruction imparted to the students of—and the class of students taught by—these Schools, in order to afford a clear understanding of their salient and unique features.

The only requirement for admission to any of the courses offered by the International Correspondence Schools, is that the applicant shall be able to read the English language and to write it sufficiently well to make his written answers to the questions asked him intelligible. Each course is complete in itself, and no textbooks are required other than those prepared by the Schools for the particular course selected. The students themselves are from every class, trade, and profession and from every country; they are, almost without exception, busily engaged in some vocation, and can spare but little time for study, and that usually outside of their regular working hours. The information desired is such as can be immediately applied in practice, so that the student may be enabled to exchange his present vocation for a more congenial one, or to rise to a higher level in the one he now pursues. Furthermore, he wishes to obtain a good working knowledge of the subjects treated in the shortest time and in the most direct manner possible.

In meeting these requirements, we have produced a set of books that in many respects, and particularly in the general plan followed, are absolutely unique. In the majority of subjects treated the knowledge of mathematics required is limited to the simplest principles of arithmetic and mensuration, and in no case is any greater knowledge of mathematics needed than the simplest elementary principles of algebra, geometry, and trigonometry, with a thorough, practical acquaintance with the use of the logarithmic table. To effect this result, derivations of rules and formulas are omitted, but thorough and complete instructions are given regarding how, when, and under what circumstances any particular rule, formula, or process should be applied; and whenever possible one or more examples, such as would be likely to arise in actual practice—together with their solutions—are given to illustrate and explain its application.

In preparing these textbooks, it has been our constant endeavor to view the matter from the student's standpoint, and to try and anticipate everything that would cause him trouble. The utmost pains have been taken to avoid and correct any and all ambiguous expressions—both those due to faulty rhetoric and those due to insufficiency of statement or explanation. As the best way to make a statement, explanation, or description clear is to give a picture or a diagram in connection with it, illustrations have been used almost without limit. The illustrations have in all cases been adapted to the requirements of the text, and projections and sections or outline, partially shaded, or full-shaded perspectives have been used, according to which will best produce the desired results. Half-tones have been used rather sparingly, except in those cases where the general effect is desired rather than the actual details.

It is obvious that books prepared along the lines mentioned must not only be clear and concise beyond anything heretofore attempted, but they must also possess unequaled value for reference purposes. They not only give the maximum of information in a minimum space, but this information is so ingeniously arranged and correlated, and the

indexes are so full and complete, that it can at once be made available to the reader. The numerous examples and explanatory remarks, together with the absence of long demonstrations and abstruse mathematical calculations, are of great assistance in helping one select the proper formula, method, or process and in teaching him how and when it should be used.

This volume treats of water supply, sewerage, and irrigation. The subjects of water supply and sewerage are treated very fully, both from the engineering and from the sanitary point of view. Much of the volume is devoted to the purification of water and the treatment and disposal of sewage. In connection with water supply, all kinds of pipes used, including riveted and stave pipes, are fully described. The design and construction of reservoirs, standpipes, and elevated tanks are given at great length; and all details and accessories—gates, valves, manholes, catch basins, etc.—of pipe lines and of sewers are described and illustrated in their proper places.

The method of numbering the pages, cuts, articles, etc. is such that each subject or part, when the subject is divided into two or more parts, is complete in itself; hence, in order to make the index intelligible, it was necessary to give each subject or part a number. This number is placed at the top of each page, on the headline, opposite the page number; and to distinguish it from the page number it is preceded by the printer's section mark (§). Consequently, a reference such as § 16, page 26, will be readily found by looking along the inside edges of the headlines until § 16 is found, and then through § 16 until page 26 is found.

INTERNATIONAL TEXTBOOK COMPANY

CONTENTS

WATER SUPPLY	*Section*	*Page*
General Considerations	84	1
Quantity of Water	84	4
Variations in Water Consumption	84	10
Empirical Formulas for Water Consumption	84	14
Rainfall and Its Relation to Water Supply	84	18
Streams	84	25
Wells	84	32
Springs	84	46
River and Lake Intakes	84	50
Practical Applications of Hydraulics	84	52
Pipe Branches	84	54
Compound Pipe Line	84	59
Roughness of Pipes	84	62
Methods of Distribution	85	1
Design of a Pipe System	85	5
Construction of Pipe Lines	85	13
Pipe Laying	85	13
Special Castings	85	29
Valves	85	31
Meters	85	35
Hydrants and Fire-Hose	85	38
Cast-Iron Pipe	85	43
Steel Pipe	85	47
Wooden-Stave Pipe	85	51
Pumping	85	55
Reservoirs	86	1

	Section	Page
WATER SUPPLY—*Continued*		
Location and Capacity of Standpipes	86	19
Design and Construction of Standpipes	86	27
Anchorage and Foundation for Standpipes	86	36
Elevated Tanks	86	45
SEWERAGE		
Preliminaries	87	1
Surveys	87	4
Maps	87	8
Grades	87	11
Rate of Rainfall	87	15
Maximum Rate of Flow	87	28
Quantity of Sewage in Separate System	87	47
Arrangement of Plans for Sewerage Systems	88	1
Sewer Computations	88	9
Egg-Shaped Sewers	88	20
Vitrified Terra-Cotta Pipe	88	28
Brick and Concrete Sewers	88	26
Cement, Wood, and Iron Sewers	88	40
Manholes and Lamp Holes	89	1
Catch Basins	89	11
Foundations for Sewers	89	12
Construction of Brick and of Concrete Sewers	89	16
Stream and Other Crossings	89	19
House Connections	89	24
Brick-Sewer Intersections	89	26
Overflows	89	28
Outlets	89	31
Trenching for Sewers	89	33
Flush Tanks	89	38
Sewer Surveys and Records	89	40
Estimates of Materials and Cost	89	47
PURIFICATION OF WATER		
Impurities in Water	90	1
Hardness	90	11

Purification of Water—*Continued*	Section	Page
Examination of Water	90	14
Chemical Analysis	90	17
Bacteriological and Microscopic Examinations	90	23
Design of Settling Basins	90	25
Coagulation	90	30
Water Filtration	91	1
Construction of Slow Sand Filters	91	4
Mechanical Filters	91	23
Miscellaneous Matters Relating to Filters	91	29
Purification by Copper Sulphate	91	31
Removal of Iron from Ground Water	91	33
Softening of Hard Water	91	34
Sewage Purification and Disposal		
Composition of Sewage	92	1
Interpretation of Sewage Analyses	92	5
Dissemination of Infectious Diseases	92	13
Tests of Sewage Effluent	92	16
Discharge of Sewage into Streams and Lakes	92	16
Chemical Precipitation	92	19
Biological Purification	92	24
Broad Irrigation	92	28
Intermittent Filtration	92	39
Sedimentation and Septic Action	92	44
Oxidation in Porous Material	92	50
Summary	92	62
Irrigation		
Introduction	93	1
Quantity and Quality of Water	93	5
Sources of Supply	93	11
Water Storage	93	21
Dams	93	23
Canals	93	28
Earthen Canals	93	31
Lined Canals	93	35

Irrigation—*Continued*	*Section*	*Page*
Flumes	93	45
Pipes and Tunnels	93	53
Methods of Irrigation	93	54
Gauging Water for Users	93	59
Crops	93	62
American Laws on Irrigation	93	65

WATER SUPPLY

(PART 1)

GENERAL CONSIDERATIONS

1. Advantages of an Adequate Water Supply.—In considering the question of the supply and distribution of water, it is necessary to have a full understanding of the end in view, and of the requisite means to attain that end. In thinly populated localities, as in country districts, a water supply for the separate houses is found in wells and springs, and frequently the location of a house is determined by the ease and availability of such a supply. When, however, owing to an increase in the density of population, wells begin to be polluted or fail to deliver the required supply, it becomes necessary to discard these primitive methods and to search for other sources and methods that will supply enough water for the needs of the community.

All cities, as they grow, experience the need of improving and enlarging their water supply, and it is wise for a city to recognize this fact, and to provide, in its early days, such facilities for the development of its water supply that all parts of any system, as they are installed, may serve to form parts of an ever-expanding whole.

Aside from the convenience arising from an unfailing supply of water, one of its greatest advantages is the aid it gives in reducing losses from fire. The old bucket brigade —bailing water from a well, and passing the buckets laboriously by hand to the roof of a burning building—has given way in most cities to fire-engines, by which, through large

hose, streams of water can be turned on a fire, even in high buildings. A well-organized fire department can have water drenching a fire within a few moments after the fire is discovered. This method of fire-fighting causes a lowering in the insurance rates from about 40 cents per $100 to about 20 cents per $100. If a man that owns a house worth $5,000 must pay $20 a year for insurance when the only protection against fire is a well, he will have to pay but $10 per year when a water supply and fire department are introduced, The money that he saves in insurance he may devote to paying for the water supply, the conveniences and advantages of which are in every respect far superior to those of a well.

Another great advantage of a water supply is that it renders practicable the replacing of privies by water closets. No way of carrying off the wastes of a house has been found equal to washing them into some stream or lake. This addition to the comfort and health of the household would in itself justify the introduction of a public water supply, notwithstanding the fact that this use of water involves the construction of a special system of pipes to carry off the wastes.

Again, an abundant supply of water makes possible the establishment of such industries as sugar refineries, paper mills, and others that require large amounts of water. Such industries make the town in which they are located prosperous, reduce the general city taxes, and offer employment to many people. It is also obvious that a good water supply, by increasing the comforts and conveniences of ordinary life, causes a corresponding increase in the value of property.

2. Choice of a Source of Supply.—There are no fixed rules by which a selection of a source of water supply can be made, nor indeed by which any part of the system can be designed. The judgment of the engineer that plans the work—a judgment that should be based on the knowledge of certain fundamental principles, as well as on experience and common sense—must always be the main factor in all

the work involved. The engineer will inform himself as to all the possible sources of supply. He will consider carefully all the water supplies in the vicinity and for some distance away. He will view each supply in the light of three factors—quantity, quality, and cost. For example, the engineer in reporting on a proper water supply for the city of Syracuse, New York, some years ago, compared the merits of eleven possible and different sources. The reports of the engineers for the Boston water supply (1895) showed that they weighed the merits of various possible sources on fourteen different streams, extending from Connecticut, through Massachusetts, into Vermont and New Hampshire. No source is too insignificant for the engineer's attention. A stream, small and apparently unimportant in summer, may in the spring carry such a flow that, by suitable storage reservoirs, it may be made to furnish all the water required. A source of supply, apparently out of the question on account of its filthy condition, is not to be overlooked, since by the modern filtration methods a polluted water can be made purer than the average surface water. The city of Albany, New York, for instance, takes its water from the Hudson River at a point where it is very foul, and makes it more healthful to drink than the water that comes to Chicago out of the depths of Lake Michigan. Nor is it safe to discard a possible source of supply on account of its distance. The city of Los Angeles, California, approved a plan by which a wholesome water supply is obtained from the Owens River, 240 miles away. The city of New York has for many years considered the head-waters of the Hudson River, about 200 miles away, as one of its possible sources of supply.

Of the three factors mentioned, quantity is the most important. Unless there is a reasonable certainty that the amount of water necessary, not only in the present but also in the future, is available, no other considerations will make a proposed source of supply desirable, except as a temporary expedient. The other two factors are more elastic. A city may choose a water less wholesome, less clear, more likely to be polluted than another, simply because it is cheaper; or

it may prefer an impure and expensive lake water to a polluted water that must be filtered, on account of the prejudice against using a purified water from a polluted source. But, to the engineer, the best supply is that which furnishes the required quantity of water, of standard quality, at the least cost. In general, the question of cost, other things being equal, decides the choice of supply.

The subject of quantity will be briefly treated in the following articles; that of quality is treated in the Section entitled *Water Purification*. The subject of cost is beyond the scope of this work.

QUANTITY OF WATER

3. Factors Governing Quantity.—No fixed rules can be given for the quantity of water required by any community, and the only basis for estimate is the experience of other communities of the same character. The principal conditions that must be considered as affecting the quantity used are: the social state of the community, the extent of the manufacturing interests using city water, the quantity of water used for public purposes, the size of the city, the quantity of water wasted, the number of water meters installed, the probable future increase of population, and the amount required for fires.

4. Social State.—It has been shown in a discussion of the quantity of water needed for Boston that certain parts of the city use water at a rate very different from the rate that obtains in other parts, and that some of the suburbs differ from others in their use of water. This difference seems to be due largely to the luxurious manner of living of some of the residents, and the general waste that characterizes their homes.

Table I, at the end of this Section, shows the water consumption in the various classes of residences in Boston and vicinity. From this table, which is based on actual meter readings, it appears that, in the most luxurious of the high-class apartment houses in the fashionable part of Boston,

water is used at the rate of 59 gallons per head per day; that is, the total amount used in that section of the city is equal to the total population of the section, including men, women, and children, multiplied by 59. Parts of Boston that are not so fashionable use water at the rate of 46 gallons per head per day, and the table shows the gradation of consumption through the different stages down to the modest factory homes of Newton, Massachusetts, provided with a single faucet, where the consumption is but 7 gallons per head per day. It is evident, then, that the engineer, in preparing an estimate of the probable quantity of water to be used, must know the social condition of the community and be prepared to measure it, as it were, in gallons of water.

5. Amount of Manufacturing.—It is customary to divide the amount of water that a city uses for manufacturing by the number of persons in that city, and say that, for manufacturing, the city uses so many gallons per head per day. Evidently, this quotient is not a constant quantity for different cities, and it is a very unsatisfactory way of stating the amount. The quantity depends on the number of manufactories and not on the number of people. A large city with little manufacturing uses almost no water per head per day for that purpose, while cities with large industries devote a large part of the total supply of water to these industries. In Boston, the amount of water used for manufacturing in the year 1892 was 12,406,920 gallons; with a population of 921,000, this was equivalent to a consumption of 13.5 gallons per head per day. Syracuse, New York, used in 1889 about 30 gallons per head per day for similar purposes; Yonkers, New York, used, in 1897, about 27 gallons. The water for some large industries, such as the breweries in St. Louis, Missouri, may be obtained from wells, so that large industrial plants do not always require city water in great quantities.

The foregoing facts show that industries may use large amounts of water; that the water used for that purpose may be double the amount required for purely domestic purposes;

and that to predict the amount needed in the future requires a knowledge both of the growth of the industries and of the amount of water they will require.

6. Size of City.—It is probable that the size of a city is a slight factor in the amount of water used by that city per head per day. A tabulation of different cities of the United States arranged in order of their sizes shows a tendency for the consumption per head to increase as the city grows. There are, however, so many other factors of more importance that the effect of this is hardly worth considering. Table II, which is given at the end of this Section, and is taken from the Manual of American Water Works for 1897, shows the populations and the consumption per head per day in groups of cities in the United States.

7. Waste.—The important factor of waste is almost beyond the knowledge or control of the designer of any system of waterworks, and it is only possible to show that waste is large in total amount and per head, and that its effect is to require an amount nearly if not quite equal to twice the amount actually needed. Waste comes from many causes: there are always defective joints in the main; there are house connections that are broken and leak; there are fixtures, faucets, tank valves, and water closets that are not tight and allow a continuous stream of water to run to waste. There are also many householders that allow faucets to stand open and water to run freely day and night, to save the cost of repairs and avoid the inconveniences caused by freezing in cold weather. The following facts may show the importance of leakage:

In one of the towns of the Metropolitan District of Boston, there was a new waterworks system, only 4 years old. There was a way of measuring all the water that went into the mains, and, as all of the house connections were metered, except in a few cases where it was easy to estimate the flow, the difference between the reading of the main meter and the sum of the house meters, corrected for some water not metered, gave the loss of water by leakage from the mains.

The daily average in 1893 shown by the main meter was 128,560 gallons, while the average amount measured by the house meters was 65,380 gallons, which showed a loss of nearly half of all the water furnished. Similarly, in Fall River, Massachusetts, where the greatest pains was taken to prevent leakage, 37 per cent. of the water pumped could not be accounted for, except by leakage.

Conservative engineers estimate that it is not possible to reduce the amount of leakage below 1,500 gallons per mile of pipe, and that it is likely to be double that amount. If a city is well built up, there will be about 600 persons per mile of pipe, so that the leakage amounts to about 3,000 ÷ 600, or 5, gallons per head per day, as a possible minimum, while practically the minimum is found to be about three times this amount.

8. Number of Water Meters.—The average householder that has paid his water tax feels at perfect liberty to use or waste all the water he or his household can, and it is to correct this abuse that water meters have been of late years so generally introduced, as a necessary part of a water-supply system. They measure the amount of water used in the house, and that water is paid for. If the householder wishes to let the water run in winter, to prevent freezing, he must pay for it. If he is careful and uses but little water, he only pays for what he uses. The great advantage of meters is that by their use the total quantity of water consumed by a city is materially diminished. If half of the services of a city are metered, the consumption per head in that city is much less than that in a city with no meters. Table III, at the end of this Section, shows the effect of the introduction of meters in reducing the per capita consumption. This table is based on the statistics of 136 cities in the United States for 1901.

9. Summary.—Table IV, which is given at the end of this Section, is a brief summary of the amounts discussed in the above articles. The table is taken from a book entitled "Public Water Supplies," by Turneaure and Russell.

10. Probable Future Population.—In order that a water supply may be adequate to future requirements, it is necessary to estimate and provide for the total population of the town or city at some future period, up to which the supply must prove sufficient. Provision should be made for the probable population at the end of 20 years at least. The population of American cities and towns is given in the reports of the United States Census. A census is taken every 10 years by the United States Government, and the reports may be had for the asking from Washington. Some cities have special census reports, the census being taken either by the police of the city or by the school board. Sometimes, these censuses are taken in 10-year periods to alternate with those of the national government, and sometimes they are taken every 3 years.

If the rate at which the population increases is known, the population at any future period can be easily computed. It should be understood, however, that the result of the computation is only the *probable* approximate value of the population, since the rate of increase is never absolutely uniform.

Let r be the rate of increase per inhabitant per year; that is, let the increase at the end of a year be equal to r times the population at the beginning of the year. Then, if the population at the beginning of any given period is P_0, the population P_1 after a period of 1 year will be given by the formula

$$P_1 = P_0 + P_0 r = P_0(1 + r)$$

Similarly, for the population after an interval of 2 years,

$$P_2 = P_1(1 + r) = P_0(1 + r)(1 + r) = P_0(1 + r)^2$$

For the population after a period of 3 years,

$$P_3 = P_2(1 + r) = P_0(1 + r)^2(1 + r) = P_0(1 + r)^3$$

In general, the population after a period of n years is given by the formula

$$P_n = P_0(1 + r)^n \qquad (1)$$

To determine the value of $(1 + r)$, the population (P_0) at a certain period is taken from the census; also, the population (P_n) a certain number of years later. Knowing

P_0, P_n, and n, formula **1** can be solved for $1 + r$. This gives

$$1 + r = \sqrt[n]{\frac{P_n}{P_0}} \qquad (2)$$

When there are data from which to compute several values of r, as when three or more census returns are available, a mean of the different values may be adopted. If, however, r constantly decreases or constantly increases, it is safer to take the value obtained from the two most recent censuses.

EXAMPLE.—If the population of Scranton, Pennsylvania, by the 1890 census was 75,220, and by the 1900 census, 102,020, (*a*) what is the rate of increase? (*b*) what will be the probable population in 1920?

SOLUTION.—(*a*) Here, $P_0 = 75{,}220$, $P_n = 102{,}020$, and $n = 10$. Substituting these values in formula **2**,

$$1 + r = \sqrt[10]{\frac{102{,}020}{75{,}220}}, \text{ whence } r = .031. \text{ Ans.}$$

(*b*) In this case, $P_0 = 102{,}020$, $r = .031$, and $n = 20$. Substituting in formula **1**,

$$P_n = 102{,}020\,(1 + .031)^{20} = 187{,}710. \text{ Ans.}$$

11. Amount Required for Fires.—Table V, at the end of this Section, shows the estimate of eminent engineers as to the number of fire streams required in cities of various sizes, each stream being understood to throw 200 to 250 gallons per minute. The estimate is based not merely on the individual judgment of these men but also on the experience of fire-department chiefs in about fifty American cities. The effect of this requirement on the total water consumption may be best seen from an example. A city of 10,000 may be assumed to use water at the rate of 75 gallons per head per day, and the maximum consumption may be twice that amount, or 150 gallons per head, or a rate of 1,500,000 gallons per day. Should a large fire occur in such a city, there would be needed nine streams at the rate of 250 gallons per minute, or an additional amount of 135,000 gallons per hour. This additional amount may be needed for a period of 6 hours, during which time the maximum amount of water may also be needed for domestic purposes. The sum of these amounts gives a daily rate of 1,500,000 gallons for domestic

purposes, and 3,240,000 gallons for fire purposes, the total being about 300 per cent. of the maximum daily domestic consumption. On the other hand, if the city has a population of 100,000, the same computations show that the total requirement is only about 150 per cent. of the maximum domestic requirement. For small cities, the size of mains, the capacity of storage basins, and the duty of pumps must in general be designed for the requirements of fire streams, and if large enough for that purpose, it may be assumed that they will carry enough water for ordinary domestic needs.

VARIATIONS IN CONSUMPTION

12. Actual Variations.—It would seem from the foregoing discussion that the daily average consumption per head might be approximated from a knowledge of the local conditions; but if the published statistics of the various cities having waterworks are consulted, it will be found that there exists a wide variation in the consumption per head per day—so wide as almost to defy explanation. The amount ranges from 20 to 275 gallons per head per day. Table VI, at the end of this Section, shows the amounts used in different cities in 1897. The high averages shown in the first column of the table are undoubtedly due to extravagant waste and to the low percentage of metered services, but the table also shows the uncertainty in predicting the rate of consumption in any city, since there is no apparent reason why two cities such as Allegheny in Pennsylvania, and Providence in Rhode Island, of about the same size, should differ so materially in water consumption.

13. Variations From the Average.—More elaborate studies of consumption show that the rate varies not only among different cities, but also from time to time in the same city. These variations are caused by variations in the demand for water, which depend largely on the domestic habits of the people. As much water is used in some cities for sprinkling lawns, the consumption increases largely during the summer months. In some cities a great deal of water is used by

mills. At the noon hour, when work ceases, there is relatively little consumption. It is necessary to consider these variations in order that there may not be a deficiency in certain months or days, or at certain hours.

14. Monthly Variations.—From the records of thirteen American cities and towns, the average daily consumption for each month, expressed as a percentage of the daily average consumption for the year, is found to be as given in Table VII, at the end of this Section. It should be observed that, in systems depending on the ordinary flow of a river or stream, the fluctuations in flow must be taken into account. In the 5 months when the flow of the stream is naturally at a minimum, the average daily draft is about 12.5 per cent. in excess of the average for the year. In designing storage systems, according to the methods given later, this factor should be provided for by the construction of storage reservoirs.

15. Daily Variations.—Owing greatly to the habits of the residents of a city, the consumption from day to day varies, although in cities where the waste and leakage are large the variation is not always appreciable. Table VIII, which is given at the end of this Section, and is taken from Turneaure and Russell, shows the relation of both the maximum monthly and the maximum daily consumption to the daily average. This table shows that the maximum daily consumption may be nearly twice the average daily consumption, though it will be usually safe to take the maximum as 1.5 times the average.

16. Hourly Variations.—The quantities given for daily consumption are averages; and, since at night there is much less consumption than through the day, there must be an excess of water used during a few hours of the day. Wherever there is large leakage from the mains, the ratio of night flow to day flow is nearer unity than in those cities where little leakage exists. In fact, if there were no leakage, it might be expected that there would be practically no consumption from about midnight to 5 o'clock in the morning.

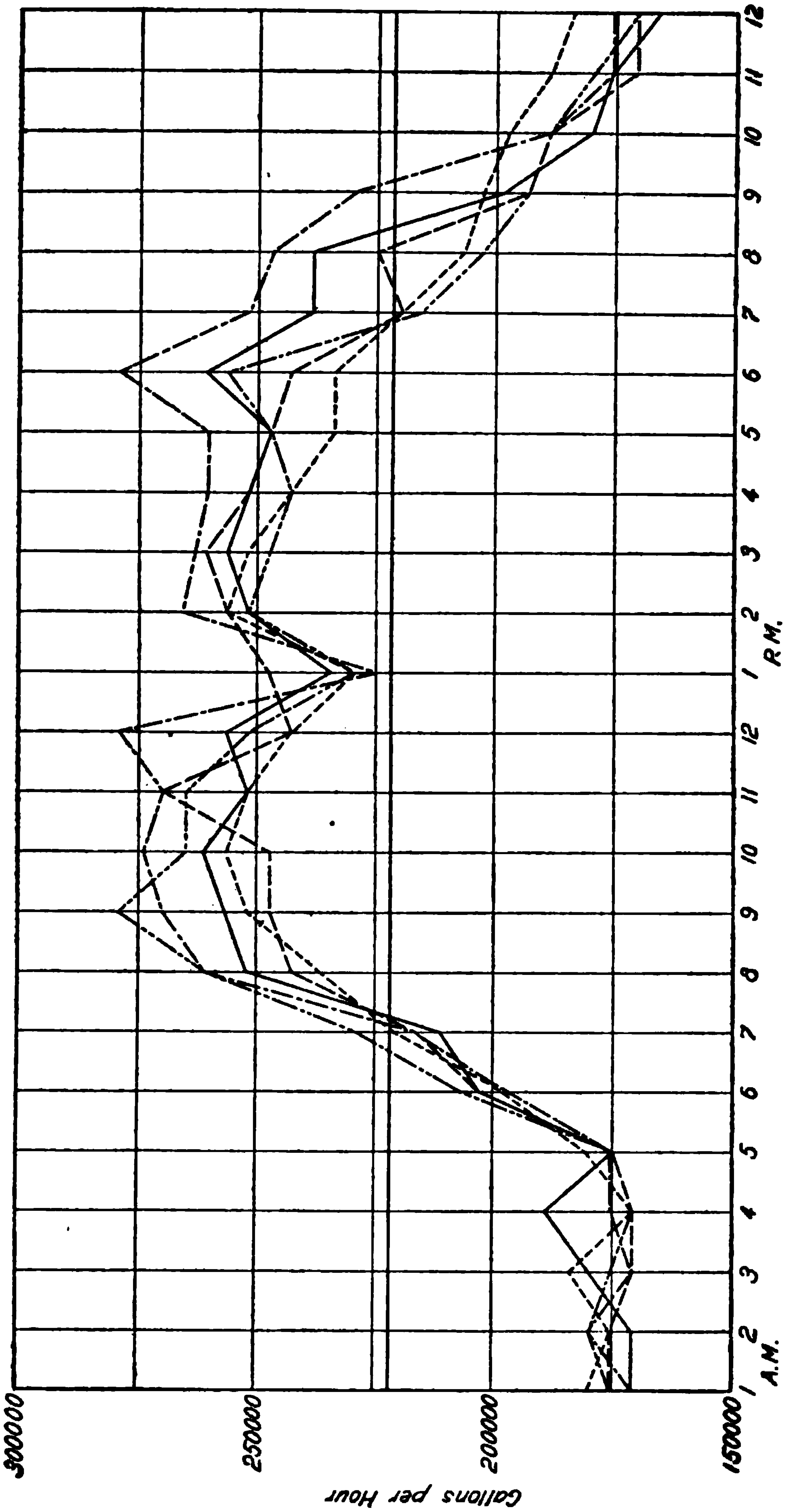

300000
250000
200000
150000
Gallons per Hour
1 2 3 4 5 6 7 8 9 10 11 12
A.M.
1 2 3 4 5 6 7 8 9 10 11 12
P.M.

FIG. 1

Unfortunately, this condition does not exist in any city, as there is always more or less leakage. In a general way, it may be said that the rate of flow for the 6 hours in the middle of the day is twice the average flow for the 24 hours. Fig. 1, taken from Ogden's "Sewer Design," shows for the 5 days for which it is drawn the hourly variations of consumption for the city of Binghamton, New York, from August 9 to August 13, 1897. The average amount for the 5 days is 220,444 gallons per hour, and the maximum daily average is 272,400 gallons per hour. The minimum amount from 11 P. M. to 5 A. M. is about 175,000, and the maximum on three of the days reaches 275,000 or more. This is an unusually small difference, and represents probably the least difference between night and day flow that can be expected. On the other hand, in Attleboro, Massachusetts, it was found that the rate, on the maximum day of the maximum month, was 155 per cent. of the average, and that on that day the maximum hourly rate was 333 per cent. of the daily average for the year, or the maximum hourly rate on that day was 215 per cent. of the average on that day. It is further to be noted that, for 8 hours in the middle of the day, the consumption is in excess of the average, and storage designed for day supply, for example, must be designed to hold, not the product of 8 and the average consumption per hour, but the product of 8 and the maximum hourly rate, which is usually taken as 1.5 times the average.

17. Summary of Discussion of Variations.—It has been shown that, while the daily average consumption for any city may be made up by adding the amounts used for domestic, manufacturing, and public purposes, and the amounts lost by waste and leakage, yet the knowledge required for an intelligent determination of these amounts is such that the proper sum is uncertain. It has been shown that actually the amount of water used in various cities of the United States ranges from 20 to 275 gallons per head per day. It has been further shown that these amounts are the average rates of consumption, day by day, throughout

the year. These averages are affected by variations in the consumption on account of the conditions of the weather as well as of the habits and occupation of the people, such variations causing great differences between the maximum values and the average values. Pipes and reservoirs should be designed for a capacity of at least twice the daily average for the year.

EMPIRICAL FORMULA FOR WATER CONSUMPTION

18. From statistics covering a wide range of conditions and embracing a period of many years, F. C. Coffin has constructed the following empirical formula for the proper amount G_d (gallons per head per day) of water to be provided for domestic and commercial uses:

$$G_d = 40\,P^{.14} \qquad (1)$$

In this formula, P is the population of the town or city, in thousands. Thus, if the population is 150,000, the value of P is 150.

The number of fire streams F that may be called into use at any one time is best expressed by the formula of Kuichling:

$$F = 2.8\sqrt{P} \qquad (2)$$

Each fire stream is assumed to have a discharging capacity of 250 gallons per minute.

If the maximum hourly rate during the business hours of the day is used, instead of the average rate per day, and if this maximum hourly rate is taken as 1.5 times the average hourly rate computed from the daily average, formula **1** may be transformed as follows:

Since P is the number of thousands of population, 1,000 P is the total population; and since, by formula **1**, $40\,P^{.14}$ is the number of gallons per head, $1{,}000\,P \times 40\,P^{.14}$, or $40{,}000\,P^{1.14}$, is the total consumption in gallons per day. Dividing by 24×60 to reduce to gallons per minute, and multiplying by 1.5 for maximum rate, we get, for the number of gallons per minute,

$$G_m = \frac{40{,}000\,P^{1.14} \times 1.5}{24 \times 60} = 41.66\,P^{1.14} \qquad (3)$$

Combining this formula with formula **2**, and noting that, since F is the number of fire streams, 250 F is the number of gallons per minute, we obtain, for the total discharge G per minute,

$$G = 41.66\,P^{1.14} + 700\sqrt{P} \qquad (4)$$

Table IX, at the end of this Section, has been computed by the use of these formulas, and offers a quick method of making an approximate estimate of the amount of water required in any city whose population does not exceed 200,000.

EXAMPLE.—A city with a population of 36,000 has a pumping plant with a maximum capacity of 3,750 gallons per minute. (*a*) Is this supply adequate? (*b*) How many fire streams can this supply furnish?

SOLUTION.—(*a*) To apply formula **4**, we have $P = 36$. Substituting this value in the formula,

$$G = 41.66 \times 36^{1.14} + 700\sqrt{36} = 6{,}677 \text{ gal. per min.}$$

It is thus seen that the supply is less than the total amount required.

(*b*) The amount of water required for domestic use is found by formula **3**:

$$G_d = 41.66 \times 36^{1.14} = 2{,}477 \text{ gal. per min.}$$

The capacity of the pumps is 3,750; therefore, $3{,}750 - 2{,}477 = 1{,}273$ = number of gallons per minute available for fire purposes. The number of streams is $\frac{1{,}273}{250} = 5$. Ans.

EXAMPLES FOR PRACTICE

1. The population of a city being 70,000, how many million gallons per day will be a safe provision for domestic and fire purposes? (Give answer to the nearest million.) Ans. 16 million gal.

2. (*a*) Determine, by means of Table IX, the safe allowance for domestic and fire purposes for a city whose population is 25,000. (*b*) What is the number of fire streams that should be available at one time? (Use interpolation.) Ans. $\begin{cases} (a) & 5{,}103 \text{ gal. per min.} \\ (b) & 14 \end{cases}$

3. How many gallons per head per day should be provided for in designing a water-supply system, when considering the domestic consumption, for a city of 17,000 population? Ans. 59.5 gal.

4. Determine, by means of Table IX, the total amount of water to be allowed for a city whose population is 90,000.
Ans. 30.5 cu. ft. per sec.

SOURCES OF SUPPLY

RAINFALL AND ITS RELATION TO WATER SUPPLY

19. Introductory.—Having determined the amount of water required for any community, the next step is to consider the sources of supply and the possibilities as well as the limitations of those sources. Water is being continually raised by evaporation from the ground, from vegetation, from the surface of streams, lakes, etc. This water is again precipitated to the earth in the form of rain or snow. Some of the rain is directly evaporated again; some is taken up by plants and animals; some flows over the surface into streams and rivers; and a large quantity sinks into the ground, where it forms subsurface reservoirs and streams. Many of these underground streams discharge into surface streams or lakes, or reappear in the form of springs. All sources of water supply are dependent on rainfall, and it is therefore of importance to study that phenomenon in its relation to the different sources from which water can be obtained.

20. Gauging Rainfall.—Rainfall is measured by means of an apparatus called a **rain gauge.** This apparatus may be either very rude or so carefully and accurately made as to be classed with instruments of precision. For observations made for the purpose of irrigation in districts of very light rainfall, it is desirable that the most perfect rain gauge should be used. Such gauges are furnished, with full directions, by dealers in scientific instruments. Frequently, however, it becomes necessary to commence the observations before a proper outfit can be procured. In such cases, a home-made contrivance may be used, and this may vary from a simple pail or tub set out of doors to a more elaborate apparatus.

A good rain gauge may be made by any handy tinsmith. The main difficulty in measuring rainfall consists largely in the fact that there are many light showers, in which the depth over a given area is so small as to render its measurement very uncertain. Recourse, therefore, is had to the principle of the "exaggerated scale." Fig. 2 represents the elevation and plan of a tin rain gauge, consisting of a narrow tube capped by a wide funnel. The dimensions being as given, it is evident that rain falling on the mouth of the funnel, which is a circle 12 inches in diameter, and being collected in the cylindrical vessel beneath, of which the diameter is 4 inches, will stand in the latter nine times deeper than the same volume spread over the greater area of the funnel, because the respective depths are inversely as the squares of the diameters, the ratio in this case being $\frac{12^2}{4^2} = 9$. If, therefore, after a fall of rain, water stands in the cylinder at a depth of $3\frac{5}{16}$ inches, this will indicate a precipitation of $3\frac{5}{16} \div 9$, or .368, inch. It is advisable to have a rule marked with inches divided decimally for the purpose of measuring depths. The apparatus shown in the figure can be supported in any suitable manner. It is generally placed in a cylindrical vessel of a diameter such that its edges support the sloping sides of the funnel. If used in a district subject to heavy rainfalls, the cylindrical receptacle should have a greater length than that shown in the figure.

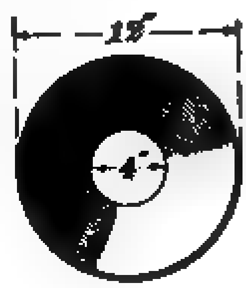


FIG. 2

Some judgment must be exercised in placing the gauge in a suitable location to secure average results. A wide, level, open space is preferable, and the mouth of the gauge should be about a foot above the general surface ot the ground. It is very desirable to have two such gauges, or even more, placed in different localities, and at

different elevations in order to guard against merely local conditions.

Measuring the equivalent precipitation of snow is more difficult. A fairly good way is to select a spot where the snow has an average depth, and invert the funnel over it; then take up the snow which it covers, melt it, and pour the resulting water into the cylinder, where it can be measured just the same as rain.

21. Quantity of Rainfall.—The quantity of rain that falls at any one place during any one year or month cannot be accurately known in advance, and it is only possible to approximate the amount by referring to the accumulated information of the past. One of the departments of the United States Government, the Weather Bureau, has, among its other duties, the task of collecting and analyzing such data, and from that department the engineer can obtain very valuable information. Its central office is in Washington, where data are received from observers stationed at different places throughout the country. The usual method of stating the rainfall is to express it as so many inches, meaning the depth to which the surface of the earth would be covered, provided none of the water flowed off. The total annual amount of rainfall depends on climatic and geographical conditions, and is, therefore, exceedingly variable.

The sea is the main source of evaporation. The general trend of the winds and the direction of the low barometric-pressure areas is from west to east across the country. These movements of pressure areas carry with them wind motions that advance from west to east, starting in the Pacific Ocean; as the warm winds holding large amounts of watery vapor strike the cold mountain ranges of the western coast, they part with the vapor in the form of rain, and the largest amount of rain is therefore found on the Pacific coast, on the western side of the coast range. Passing on east, there exists, on the east side of the mountains, flat, warm areas where water is taken from the earth rather than given to it, and the result is a succession of great desert

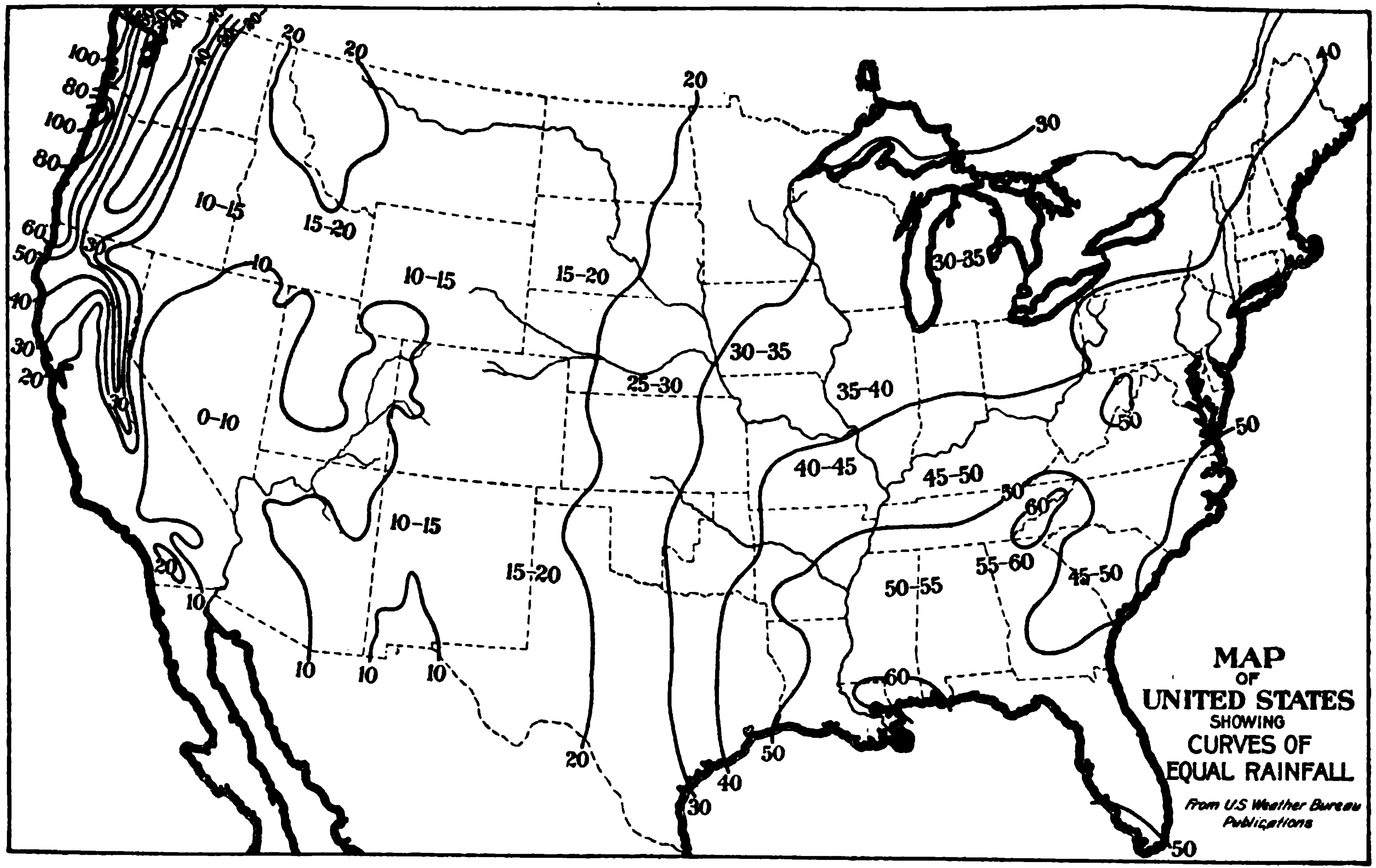

MAP
OF
UNITED STATES
SHOWING
CURVES OF
EQUAL RAINFALL
From U.S. Weather Bureau Publications
0-10
10-15
15-20
25-30
30-35
35-40
40-45
45-50
50-55
55-60

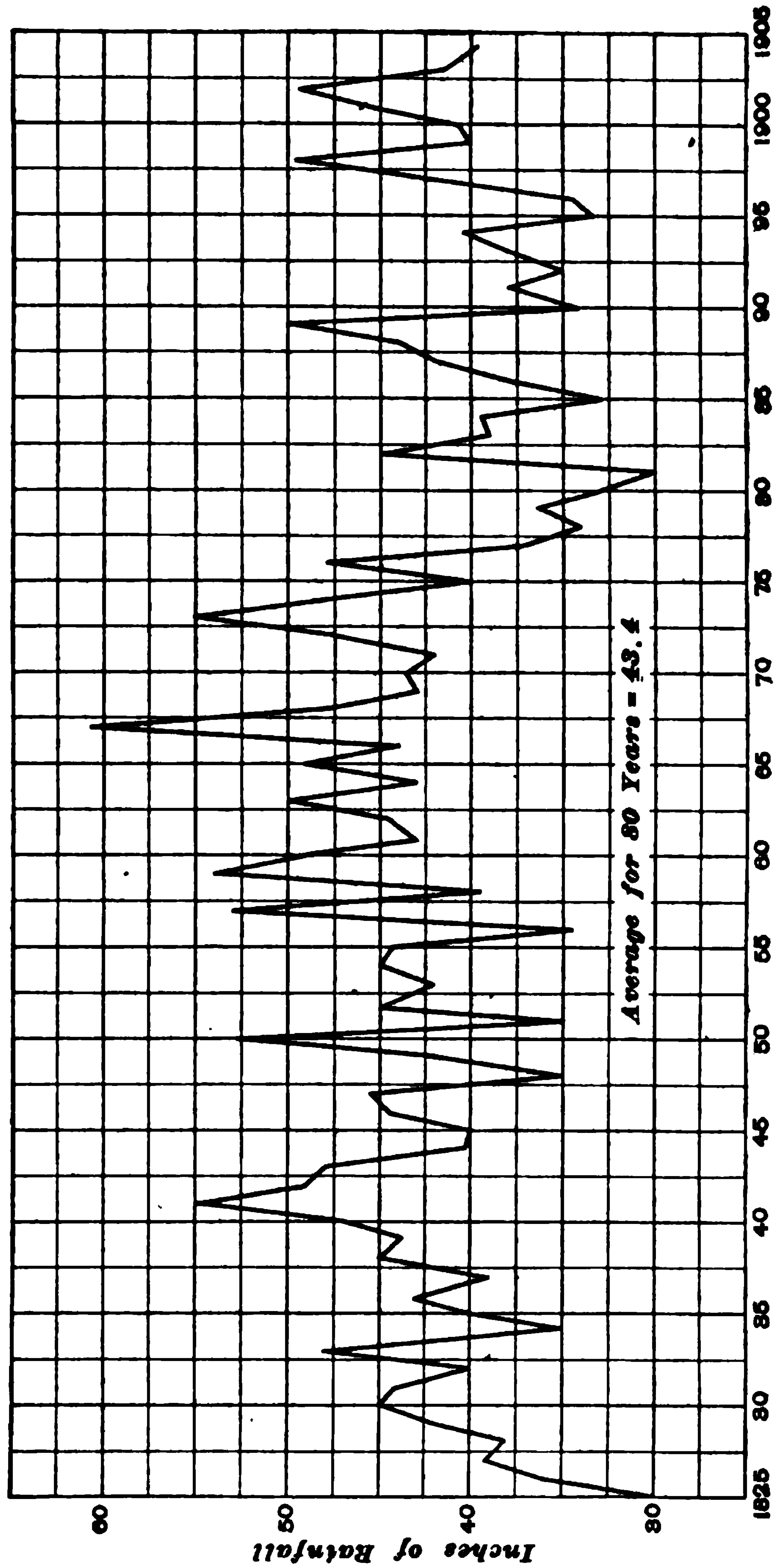
Fig. 3

areas. Finally, evaporation begins again, and a moderate rainfall is to be found up to the Mississippi valley. Here the rainfall is increased from the presence of moisture-laden winds coming from the Gulf of Mexico, which gradually deposit their moisture as they move inland.

Fig. 3, taken from the publications of the Weather Bureau, shows the distribution and amount of annual rainfall in the United States.

22. Annual Variations.—The annual rainfall is not even approximately constant. Fig. 4 shows the variations in annual rainfall for the city of Philadelphia, where records have been kept for more than 80 years. The average is about 43.4 inches. The figure makes it plain that estimates of the amount of water coming from rainfall must not be based on a single year's rainfall, but must be determined by the average of several years. It is, however, safer to use the minimum amount recorded, although it should be observed that the year of minimum rainfall is likely to be followed and preceded by years of heavy rainfall, and it is extremely rare for a low-rainfall period to last more than 3 years.

There are very few cases where a reservoir is designed on so large a scale that the rainfall period of more than a single year has been taken into account, although the recent storage reservoirs of New York are so designed. The average precipitation is only an approximation, depending on the number of years during which the observations have been taken, and there may be a large error introduced by assuming that the average of a dozen years or so is the true average, and that the lowest rainfall during that period is the real minimum. The Boston records, lasting for more than 75 years, show that the minimum annual rainfall is 58 per cent. of the average. Philadelphia records show that its year of greatest drought was 68 per cent. of the average, and the records are of nearly the same length as those of Boston. It would be proper, therefore, to consider the year of least rainfall to have about 60 to 70 per cent. of the average, although the average is itself only an approximation.



Yearly Averages

FIG. 4

Rainfall records covering as many years as possible should be obtained, in order to determine the minimum. An engineer that has devoted much study to this matter states that "dependence can be placed on any good record of 35 years' duration to give a mean rainfall correct within 2 per cent. of the truth." For shorter periods, the percentage would be about as follows: for 5, 10, 15, and 20 years, the probable deviation from the mean rainfall would be 15, 8.25, 4.75, and 3.25 per cent., respectively.

23. Monthly Variations.—The monthly variations are of the greatest importance for the design of reservoirs where water is to be stored for use in time of drought. Such variations cannot be predicted with any accuracy. In the eastern part of the United States, the months of June and September are generally the months of least precipitation; the average for Boston for the former month is 3.27 inches, and for the latter 3.55 inches. The rainfall, however, has been as high as 8.01 inches in June, and 11.95 inches in September. Similarly, while, in Boston, the two months of highest rainfall are March and August, with averages of 4.36 inches and 4.39 inches, respectively, there have been years when the rainfall has been as low as .96 inch in March, and .34 inch in August. It has been the custom in textbooks to give ratios by which the average annual rainfall could be multiplied to obtain the monthly fall, but this can be at best only a very crude approximation, and may lead to serious error in design. Table X, which is given at the end of this Section, and is taken from Folwell, shows the average, the maximum, and the minimum monthly rainfall for four cities in different parts of the United States, and will serve to show both the small difference in the monthly averages for the different months and the great differences in different years from these averages. This table also shows that for Boston and Philadelphia the average rainfall for the different months is much alike. In Boston the greatest difference between the lowest average month, June, and the highest average month, August, is only 1.12 inches. But the difference

between the rainfalls of different years is very great; the records show that in Boston the difference has been as much as 11.76 inches, and in Philadelphia, 14.22 inches. The table does not show whether months of low rainfall are likely to come together, and this is of prime importance in filling a reservoir. Fortunately, Weather-Bureau statistics show that 2 months of minimum precipitation are not likely to follow each other, and the average of 3 consecutive months, including the minimum, is generally two-thirds of the monthly average for that year. In those parts of the country, however, where there are two seasons, a wet and a dry season, there may be 4 or 5 months when the average for the whole period is much less than the monthly average for the year.

24. Evaporation and Percolation.—Evaporation occurs at all times from surfaces exposed to the air; and, in the case of large reservoirs, the amount of evaporation during the period of storage is of much importance. Evaporation from water surfaces has been carefully studied, and the amount of water lost in this way can be predicted with reasonable certainty. Evaporation takes place from the surface of the ground also, and from vegetation during its period of growth. The amount of water lost from the ground from these last causes is not so well known. Table XI, at the end of this Section, gives values for monthly evaporation in inches of depth, as determined by experiments made at Boston and Rochester, New York, by FitzGerald and Kuichling, respectively. These figures apply to evaporation from an exposed surface, such as that of a reservoir. The table shows that, during the summer months, when water is being held in storage, a large quantity is lost by evaporation. If the reservoir covers an area of 500 acres, or 21,780,000 square feet, the evaporation during June, July, and August will be about 16 vertical inches, or $1\frac{1}{3}$ feet; the volume lost by evaporation is, therefore, $21,780,000 \times 1\frac{1}{3}$, or 29,040,000, cubic feet, which is equivalent to about 2,500,000 gallons per day. This amount must be included in the consumption for which the reservoir is designed.

Evaporation from the ground depends on the physical condition of the soil, on the amount of water near the surface, and on the character of the vegetation. The proportion of the rainfall that soaks or percolates into the ground varies with the nature of the soil; it is much greater in sandy or peaty soil than in ordinary soil, and very much smaller in clay. Under favorable conditions, the percolation in ordinary soil may be 30 per cent. of the rainfall; in chalk, it may be nearly 40 per cent.; while in sand or gravel, it may be over 80 per cent.

Observation indicates that, where the area on which rainfalls is covered with vegetation, a large amount of water is absorbed by the plants, and on areas that are all planted, it may happen that no water is furnished to streams, because of the demands of vegetation. Grass, for example, on a level meadow will absorb during the summer an amount of water equal to a whole year's rainfall. Forest trees, on the other hand, absorb but very little water, and the proportion of water that can be obtained from an area covered with trees is much greater than that from cultivated soils. For this reason, as well as from the fact that the surface layer of leaves and mold in a forest retards the surface flow, allows time for percolation to occur, and increases the regularity of the stream flow, wooded watersheds are very desirable.

EXAMPLE 1.—How much water would be lost by evaporation in the month of May, out of a reservoir whose area is 1 square mile, the rate of evaporation for Boston being used?

SOLUTION.—The volume of water lost by evaporation is equal to the area of the reservoir multiplied by the depth of evaporation as given in Table XI. Using the values in the column under Boston, the volume in cubic feet is

$$640 \times 43{,}560 \times \frac{4.46}{12} = 10{,}361{,}000 \text{ cu. ft.} \quad \text{Ans.}$$

EXAMPLE 2.—(*a*) What vertical depth would be lost by evaporation from a reservoir during the months of July, August, and September, using the values for Rochester? (*b*) If the annual rainfall is 36 inches, what is the least average for these three months? (*c*) According to the New York statistics, what is the net gain or loss to the reservoir?

SOLUTION.—(*a*) Taking the values for July, August, and September from the column headed Rochester, Table XI, the vertical depth is $5.47 + 5.30 + 4.15$, or 14.92, in. Ans.

(*b*) The monthly average rainfall is $\frac{36}{12}$, or 3 in. The least average rainfall for 3 months is taken as $\frac{2}{3}$ of 3, or 2, in. Ans.

(*c*) The rainfall for 3 months is 2×3, or 6, in. The net loss to the reservoir is, then, $14.92 - 6$, or 8.92, in. Ans.

EXAMPLES FOR PRACTICE

1. What is the net loss in gallons to a reservoir approximately rectangular, 1,000 ft. × 200 ft., in Boston, from May to October, inclusive, if the annual rainfall is 40 inches? Ans. 1,923,600 gal.

2. What must be the average annual rainfall in Rochester, so that the evaporation losses may be balanced by the gain from rainfalls? Ans. 34.54 in.

STREAMS

25. Flow of Streams.—Whenever it is possible to determine the flow of a stream by actual measurements, this should be done. If the stream is small—that is, not over 50 feet wide and 2 feet deep at the time of the greatest flow—a weir should be built at some convenient locality, and

FIG. 5

daily or at least weekly observations made of the flow. In some cases, however, the construction of a weir is somewhat difficult, because the dam must be water-tight, so that the entire flow of the stream shall pass over the weir.

Sometimes, a part of the stream to be gauged may be found where it is divided into two branches by an island, as

shown in Fig. 5. In such a case, one branch may be shut off by a temporary dam TT, while the dam for the weir DD is being built. Sometimes, another temporary dam may be needed at the lower end of the island, to prevent back wash. When the weir is completed, the temporary dam is removed, and another one built across the other branch, so as to divert the water to the channel in which the weir is built.

The dam DD is built by driving sheet piling across the stream, the ends entering well into the bank on each side. For this purpose, an excavation is made in the banks; the sheet piling is then driven, and the excavation is refilled with closely packed earth. Back of the sheet piling, on the up-stream side, an embankment should be placed, the best material for which is very fine gravel or coarse sand. Clay is of very little use for this purpose, because if the least trickle of water passes through it, the clay is soon washed away, whereas the tendency of the sand and gravel is to clog any aperture that may be formed.

The methods of constructing the notch, of making the measurements, and of computing the quantity of discharge have been fully described in *Hydraulics*. When great accuracy in the quantity of discharge is not required, or where the conditions under which the observations are made, such as possible leakage around the dam, inaccuracies of measurement, or uncertainty in regard to the velocity of approach, make the result doubtful, the following formula, in which a mean value of the coefficient of discharge has been used, is more convenient than those given in *Hydraulics:*

$$Q = \frac{10}{3} b H^{\frac{3}{2}}$$

In this formula, Q is the discharge in cubic feet per second; b, the length of the weir, in feet; and H, the head over the crest of the weir, in feet.

Daily observations should be recorded for at least a year, in order to obtain a fairly approximate knowledge of the normal flow of the stream. For this purpose, the recording water gauge, described in *Hydraulics*, is very convenient.

If the stream is too large for a weir measurement, or if the time available does not permit this method of measurement, the flow may be determined by some of the other methods explained in *Hydraulics*.

In determining the flow of a stream, it is necessary to ascertain the total yearly yield, and also the minimum and maximum seasonal yields; that is, the smallest daily amount that the stream may furnish during the dry season, and the greatest that it may furnish in times of freshets. The most important element of the problem, however, is the minimum yield, both per day and per year.

26. Watershed and Rainfall.—The best way of determining the probable yield of a stream is to ascertain the area of the territory that drains into it and the amount of yearly rainfall. The area drained by the stream is called the **watershed.** It is ascertained by a survey locating the line of division between the basin of the stream under consideration and all the adjacent basins, so that the entire area over which the rainfall drains into the stream, above the site of a proposed dam, may be calculated. This survey may be tedious and difficult, and requires much care: when working at the head-waters of the different streams, the surveyor is likely to mistake the watersheds into which the different streams drain; it is frequently found that the work of several days is useless on account of the surveyor having got into the wrong watershed. Local information is very valuable to the surveyor in such cases, and should be sought and used. Extreme accuracy is not needed in this work, and in most cases a simple compass survey is all that is required, with bearings read to quarter degrees.

Fig. 6 shows the watershed of the Nashua River above the Wachusett dam of the Boston waterworks. The dotted line represents the boundary of the watershed, as determined by a survey.

When a good map of the territory drained is available, a tolerably correct estimate of the area of the watershed can be made by tracing on the map a line dividing the streams

that are tributary to the proposed reservoir from those that are not. This line will run in and out between the headwaters of the several streams, and enclose an irregular area, the extent of which can be calculated by scaling or by the use of the planimeter. Such a calculation, made on the best map procurable, will generally be the first step taken when

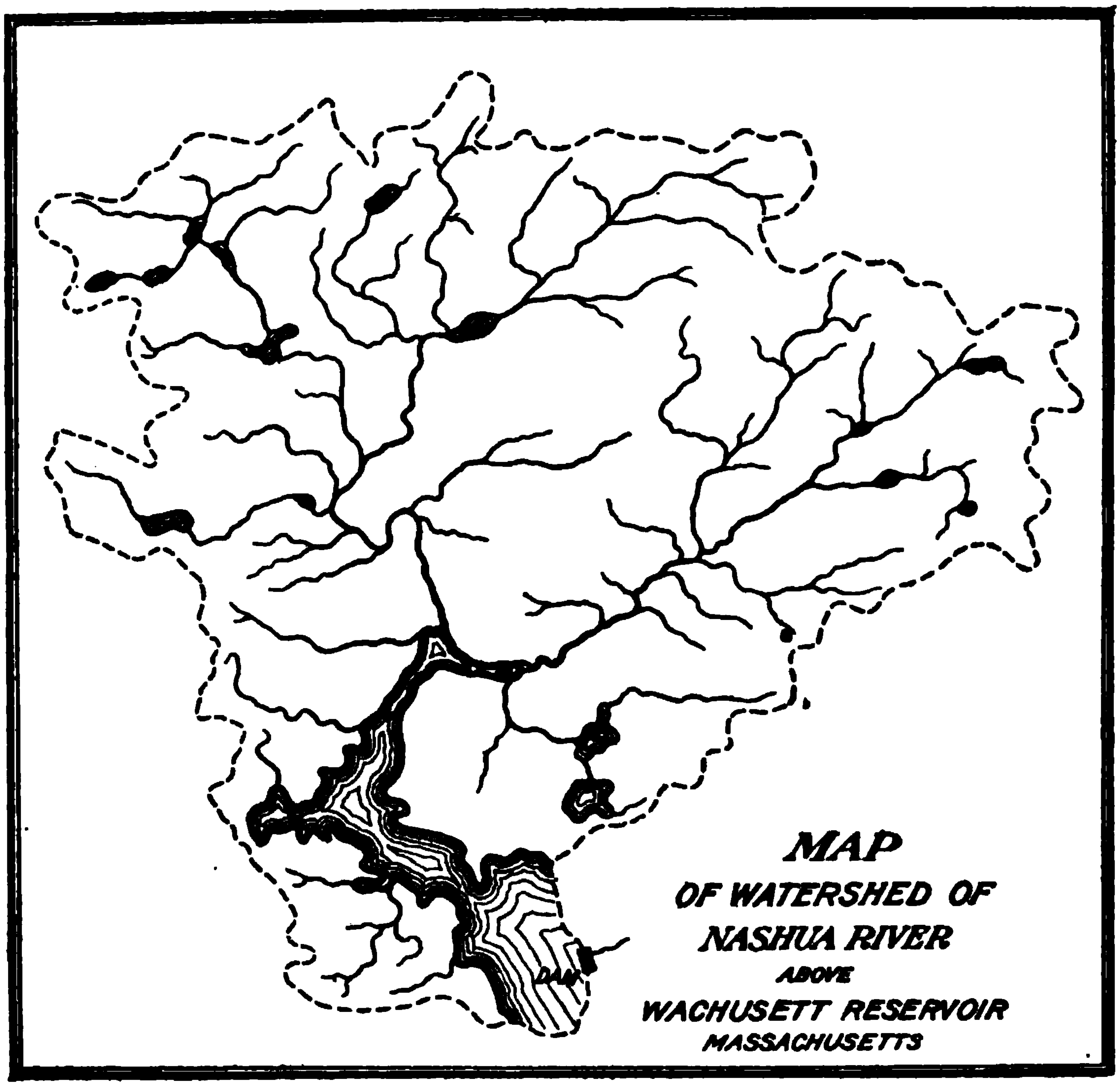


FIG. 6

comparing the advantages of several different sources of supply. The result should be verified subsequently by an actual survey.

In estimating the area of a watershed, all exposed water surfaces, such as ponds and lakes, should be deducted, because the evaporation from these will, with average rainfalls, balance the amount of precipitation on them. Even swamps and marshes, when extensive in area, should be

considered, and a certain deduction made, depending on their greater or less degree of saturation, treating them, to a corresponding degree, as exposed water surfaces.

If no records of the rainfall in the valley under consideration have been kept, the same difficulty will arise as in the case of weir measurements; that is, a great length of time will be necessary for collecting complete data. But it is very rarely that no records can be obtained, if not for the district actually studied, at least for neighboring ones, where the conditions are similar. In all cases it is advisable to commence observations of rainfall and gaugings of the stream at the same time as the survey. There is never any danger of having too many data, provided they are trustworthy and are properly used.

27. Run-Off.—The **run-off** of a basin is the quantity of water that flows into the stream draining that basin. It has already been seen that, when rain falls on a drainage area, some of it runs off directly from the surface, and some soaks into the ground. The part of the water that runs off directly, as well as that absorbed by soakage, depends on the geological, geographical, and topographical character of the basin. The run-off is that part of the rainfall which is not lost by evaporation or by percolation into subterranean channels.

In the rocky region of many of the American states, where the precipitation is from 40 to 50 inches, the run-off is frequently more than 50 per cent. of the rainfall; while in arid sandy territories, where the precipitation is but 10 or 20 inches, the run-off may be as small as 20 per cent. of the rainfall. The run-off is greatly affected by the seasons, on account of the conditions of the weather and of vegetation. From December to May, inclusive, evaporation and plant absorption are light, and a large proportion of the rainfall appears in the streams. From June to August, inclusive, vegetation is most active; frequently, not more than .1 of the rainfall appears in the streams, and generally the ground water becomes lower and lower during this period. From

September to November, inclusive, the ground water tends to recover, and a corresponding increase of the run-off is noticed.

It being understood that the run-off includes the water that runs from the surface directly and that which the stream derives from subsurface sources, it is safe to say that in the United States the run-off is from one-quarter to one-half of the annual precipitation on the watershed. The larger figure is to be taken when the basin has a hard rocky substratum, when the slopes are steep, and when the amount of the rainfall itself is an average amount. The small figure is to be taken when the ground is porous, sandy, or gravelly, and the watershed area flat and smooth, or when the rainfall itself is below the average and the demands of vegetation are large.

28. Average Flow From a Watershed.—A common available yield throughout the middle and eastern American states is 8,000,000 gallons per year per square mile of watershed for each inch of rainfall. Thus, a yearly rainfall of 46 inches will generally yield 368,000,000 gallons per year per square mile of watershed. So generally is this the case that in the Croton watershed, where the average yearly precipitation is about 46 inches, it is customary to count on an average of a million gallons per day per square mile; this corresponds to nearly one-half of the precipitation.

29. Minimum Flow From a Watershed.—The actual minimum flow of a stream gives the amount of water that can be taken from the stream without the construction of any reservoirs whatever. This minimum flow has apparently but little to do with the rainfall, but depends on the amount of spring water flowing into the stream. Table XII, which is given at the end of this Section, and is taken from the Geological Report of New Jersey, gives an indication of what may be expected from watersheds other than those mentioned, but similar in size and topography. The Tohickon and Neshaminy are streams of high slope, narrow valleys, and rocky basins; streams of this class are sometimes entirely dry. The minimum flow of the Sudbury seems to be abnormally

low for streams of its class. The average minimum flow for the other streams is seen to be about 1,000,000 gallons per day per square mile.

30. Numerical Illustration.—In order to illustrate the manner of making an intelligent study of the quantity of water that can be obtained from a source of supply, it will be best to assume a particular case. Suppose a town of 25,000 inhabitants is to be supplied with water from a stream of satisfactory quality and location, the only uncertain element of the stream being the quantity of water that it can furnish. The first thing is to decide on the quantity required. The present population of the town is 25,000. By consulting the census reports and using the method of Art. **10**, it is found that the population at the end of 20 years will probably be 50,000. The consumption, which includes that used for domestic, manufacturing, and other purposes, is assumed to be 125 gallons per head per day. If it is supposed that this rate will be maintained as the city grows, the city will, at the end of 20 years, need a daily supply of $50,000 \times 125$, or 6,250,000, gallons.

A survey has shown that the stream in question possesses a watershed of 20 square miles above the point at which the supply for the city is to be taken. Observations carried on in the neighborhood for 20 years back give an annual average rainfall of 46 inches. By Art. **22** this may be 3.25 per cent. in error, and if it is assumed to be too large—an assumption that will give too much water rather than too little—the true average may be taken as $46 - 46 \times .0325$, or 44.5, inches. The minimum rainfall, according to Art. **22**, may be 60 per cent. of the average, or $.6 \times 44.5 = 26.7$ inches.

A careful examination of the topography and geology of the drainage area leads to the conclusion that probably one-third of the rainfall, or 8.9 inches, flows into the stream. A depth of 8.9 inches over 1 square mile is equivalent to a volume of 20,676,000 cubic feet per square mile per year, or 56,648 cubic feet per square mile per day, or 423,720 gallons per square mile per day. Therefore, an area of 20 square

miles will yield 423,720 × 20, or nearly 8.5, million gallons per day. The required amount of 6.25 million gallons is, therefore, only 73 per cent. of the amount that the stream can probably furnish. Whether the minimum flow of the stream will be the required 6.25 million gallons per day or $\frac{6{,}250{,}000}{20} = 312{,}500$ gallons per square mile must be determined from a study of the stream. From Table XII, at the end of this Section, it is seen that there are a number of streams, all, however, of larger watersheds, whose minimum flow (that is, the least flow per day at any time of any year since observations were made) is greater than 312,500 gallons. There are, however, four streams in which the minimum flow is less than this amount, and it would be wise to build a storage reservoir from which to take water during the few weeks when the minimum flow may fall below the required amount.

EXAMPLES FOR PRACTICE

1. What would be the yield, in 1 year, from a watershed of 60 square miles, when the mean rainfall is 40 inches?

Ans. 19,200 million gal.

2. The minimum flow of the Ramapo River is 910,000 gallons per square mile per day. If the average rainfall is 46 inches, what is the yield in 1 year, per square mile, per inch of rainfall?

Ans. 722,070 gal.

WELLS AND SPRINGS

SOURCES OF SPRINGS AND WELLS

31. That part of the rain which sinks into the ground is to be found again, not only as an agent in maintaining a uniform flow in streams, but also in the form of springs and wells. Fig. 7 is an ideal section of a portion of the earth's crust, consisting of porous layers alternating with impervious layers. It will readily be seen that water falling on the surface *rs* will percolate to the bottom, and rise in *b*, as in a receptacle, to a greater or less height, according to its amount. This water may be reached by digging a well, and then

pumped out. A well of this kind is called a **shallow well,** or **dug well.**

Water falling at *p* and *p'* will flow to, and collect at, the bottom *c* of the porous stratum *pcp'*. It can be reached by driving a pipe through the strata *b*, *e*, and *c*, and then pumped out. This construction is called a **deep well.**

FIG. 7

Water falling at *a* will flow to *d*, and may reappear at *a'* as a **spring.** If a pipe is driven to *d*, and the pressure is sufficiently great to force the water up to the surface, an **artesian well** is obtained. As will be noticed, an artesian well is a deep well in which the water comes up by itself, instead of requiring to be pumped.

SHALLOW WELLS

32. General Considerations.—Dug, or shallow, wells (those usually found in farms) are generally 10 to 20 feet deep. The amount of water that can be obtained from a shallow well depends on the area from which the water comes, and on the size of the sand or gravel, since this size determines the velocity of flow. Shallow wells that reach only to the upper surface of the water-table and have no cover to protect them from surface impurities possess many

objectionable features that disqualify them as sources of a large supply. The most serious objection is the danger of contamination, especially in the neighborhood of human habitations. Moreover, each well furnishes only a comparatively small quantity of water, and is likely to become dry in seasons of drought, when the neighboring streams are drawing heavily on the underground storage. There are, however, some notable examples of shallow-well supplies in the United States, such as those at Waltham and Canton, Massachusetts, and at Addison, New York. If there is no reason to fear surface pollution, such a supply may be of good quality and procured at reasonable expense.

33. Construction.—As usually constructed, shallow wells are circular in form, about 12 feet in diameter on the inside, and walled up with masonry from the bottom, which is left open for the upward seepage of the water, to the top, which is suitably covered. In construction, the excavation is begun and carried down until water is struck, or until the caving of the banks is imminent. In this excavation, a well curb is constructed, as shown in Fig. 8 in plan and elevation. The curb is usually made of several thicknesses of 4″ × 8″ or 4″ × 10″ timber, breaking joints in the several tiers. For example, in constructing the bottom tier, the timbers *abdc* and *bde* would be cut to form and spiked together; in the second course, the timbers would be placed in the positions shown at *fgih* and *ghj*, etc., each course being complete, and the course above breaking joints thoroughly with the one below. An iron shoe is fastened by lag bolts to the lowest circumference, as shown. The masonry is then built up near the surface on this curb, a pump is installed to remove the water in the excavation, and men, entering the working pit, remove the earth for a foot or two beneath the bottom of the curb. The process is continued until the required depth is reached. Pipes are often placed radially around the masonry work, usually beginning one foot from the bottom, as shown in the figure, to permit a free entry of water on all sides as well as from the bottom

of the completed structure. The well, when completed, is roofed over, and the suction main of the pumping engine is placed through the wall at as low an elevation as economical

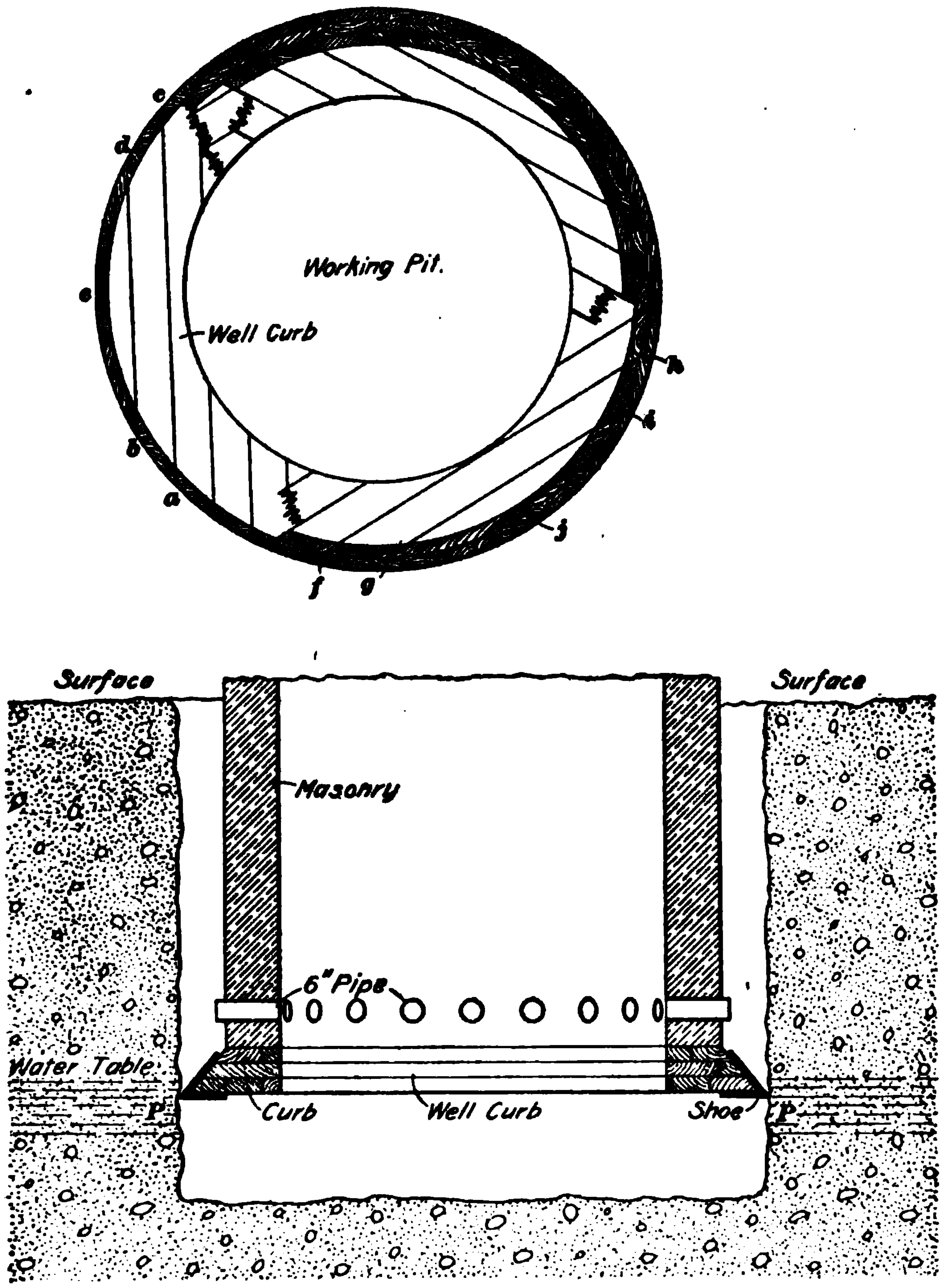


Fig. 8

operation will permit: if there is an abundance of water, a short lift of the suction should be used; but, if the quantity is doubtful, the suction may be lowered to 26 feet below the pump, although the expense of the operation will be greater.

DEEP WELLS

34. Introductory.—Deep wells are of the same general character as shallow wells, from which they differ in having the gathering ground more remote and in having impervious strata above the water-bearing soil. It often happens that several porous strata exist, from any of which water may be drawn, and the quality as well as the abundance of the flow is a factor in making the selection.

Deep wells have long been used for irrigation in the arid western regions of the United States, but it is only comparatively lately that they have been used to any great extent in the East as a source of water supply. They are destined to play a more important part in the future; for, as the country surrounding large towns becomes more and more thickly settled, the difficulty of securing large uncontaminated supplies of surface water increases, and consequently more attention must be paid to the vast quantities of pure water stored away beneath the surface of the earth. This water constitutes a body of great but unknown extent, which forms underground lakes and rivers having no doubt watersheds or areas of absorption; but the utmost uncertainty exists as to what may be the bounds of these areas.

35. Estimating the Probable Yield of a Proposed Deep Well.—In a section where no deep wells have been dug, it is difficult to predict whether or not water will be found at all within reasonable depths. The judgment of experts, based on surface indications, as to the probability of finding water in sufficient quantities in any given locality, is of small value. The most satisfactory method of determining the yield of an underground supply is to dig or drive a small test well, and by means of a steam pump attached to determine the amount of water that can be pumped. The test should be continued long enough to make sure that the supply is ample, and for a test lasting only a few days the rate of discharge should be from ten to twenty times the required rate. If the pumping is at about the rate at which

the water is to be used, perfect certainty of an adequate supply can only be obtained after a test of at least a month, which continues without lowering the water beyond a fixed minimum. The effect of such pumping can also be gauged by noting, from time to time, the depth of water in neighboring wells, assuming that if wells in the vicinity are rapidly exhausted, the body of underground water is limited and not to be depended on.

36. It is seldom that a sufficient quantity of water can be obtained from a single well; so it is customary to put in a **battery,** as a number of connected wells are collectively called. Such wells are likely to affect each other in the amount that they will discharge, and it is very important to determine the distance apart they should be in order that they will not draw too much water from one another. This may be determined by driving two wells at what is thought to be a proper distance apart (from 400 to 1,000 feet, depending partly on the depth and partly on the coarseness of the sand or gravel), and, by attaching a pump to one, finding out whether it lowers the level of the water in the other: if so, a third well is driven at a greater distance, another test is made, and the process is continued until a distance is found where no well has any appreciable effect on the others.

37. Methods of Driving Deep Wells.—In driving deep wells, the method of procedure is usually as follows: A drilling machine, Fig. 9, is placed over the point where water is desired. The drilling tool or bit, Fig. 10, is screwed into the rope socket, and the rope, after being thoroughly clamped in its socket, as shown in Fig. 11, runs over a pulley and down to the drum, on which a sufficient length of the rope is wound. By alternately raising and lowering the rope, the bit is made to strike a series of sharp blows on the earth. As blow succeeds blow, a hole is formed; when the hole is 4 or 5 feet deep, the drill is removed and a length of wrought-iron pipe, called the **casing,** is placed in the hole in a vertical position, and driven down by the

drilling machine, in the same manner as a pile is driven by a pile driver; the bit is then replaced, lowered into the pipe, the hole deepened, the case driven down, etc.

Sometimes, the pipe is kept driven a little below the point where the drill is working, and in other soils the bit works below the bottom of the pipe, and a few blows on the top of the pipe easily settle it in position. There is always some water in the bottom of the well casing; it forms a viscous, mushy mixture, which after 4 or 5 feet of drilling is removed by a **sand pump.** This pump (see Fig. 12) is a hollow tube with a flap valve in the lower end opening inwards, and with a hook on its upper end. After the bit is removed, the sand pump is lowered until it strikes the bottom of the hole; in the act of striking, the valve is forced open, and by repeated lifting and dropping the pump, it is soon filled; it is then drawn up and emptied. By alternate drilling, pipe driving, and soil removal, length after length of pipe is forced into the ground

Fig. 9

until water of satisfactory quality and in sufficient quantity is reached.

If rock is encountered during the driving, the wrought-iron pipe is driven firmly into the rock for several inches, to prevent any subsoil water from gaining access to the casing. After this is done, the driving of the casing is discontinued, and only the drilling is done. New-shaped cutting drills are introduced. **Jars** (see Fig. 13) are added, the purpose of

FIG. 10

FIG. 11

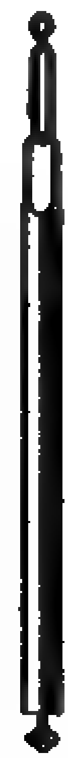


FIG. 12

FIG. 13

which is to prevent the loss of tools if from any cause they become wedged in the hole. In the figure, the two parts *a* and *b* are connected like the links of a chain. If the drill is wedged in the rock, a little slack of the rope is let off, allowing the parts *a* and *b* to hang loose, and when the engine starts, the two parts come together with a jar that usually loosens the bit, while a steady pull would not do so.

In drilling, whether in earth or rock, a man is stationed at the point where the rope emerges from the casing; one of his duties is to keep the rope rotating; he gives the rope a

twist, a dozen times to the right, and then a dozen times to the left, to avoid, as far as possible, the wedging of the drills, as well as to keep the hole circular for driving the pipe.

38. It is found that diameter has but little effect on the delivery of these wells, their yield depending rather on their depth. It is not advisable, however, to use diameters less than 4 inches, and the ordinary range runs between 4 and 8 inches. The usual practice is to commence with a larger bore than it is proposed to use for the permanent casing, and reduce the size progressively. In districts where no holes have already been put down, the probable depth, the nature of the strata, etc. are unknown, and the first holes are, consequently, more tentative than those in a well-explored region.

39. In boring a well, it is necessary to decide on the diameter of the permanent bore, and to estimate the probable depth at which water will be struck. Suppose that a 4-inch bore is required, that the depth is 1,000 feet, and that it is thought that three sizes of casing are necessary, including the final one of 4 inches; then the drilling will be commenced with a bore suitable for the reception of an 8-inch casing. After carrying this down 300 feet, it may become apparent that a smaller pipe can be used, and a 6-inch casing may be substituted. This may, perhaps, be carried down 400 feet farther, making a 6-inch casing 700 feet long, the upper part contained within the 8-inch casing. From this point it may be judged that the 4-inch casing can be used, which will then be bored for and inserted and driven down the remaining 300 feet. If convenient, the two upper sections of 6- and 8-inch pipe may now be withdrawn, leaving a continuous 4-inch pipe extending from the surface to the bottom.

The yield of these deep wells is sometimes greatly increased by exploding torpedoes at the bottom, by which means the rock is opened by fissures, allowing a freer passage for the water to reach the well. This operation requires considerable skill, and should only be attempted under the direction of an expert.

40. Methods of Delivery From Deep Wells: Suction Pumps.—When sufficient water of the required quality is found, it may happen that the well will flow, or at least that the water will come near enough to the surface to be pumped by an ordinary suction pump. This requires that the suction head should be less than 34 feet, theoretically, and practically it is found that this distance must be reduced to not more than 26 feet, in order that the pump may work. Even then, great care must be taken to have the suction pipe perfectly air-tight; otherwise, the pump will not be able to draw the water to the surface.

If the suction head is greater than 26 feet, a pit may be dug around or by the side of the well, and the pumping plant lowered until the suction pipe is of the desired length. For example, at Memphis, Tennessee, the floor of the pump pit in a battery of wells is 45 feet below the surface, and a delivery of 10,000,000 gallons per day has been secured without difficulty from eight wells.

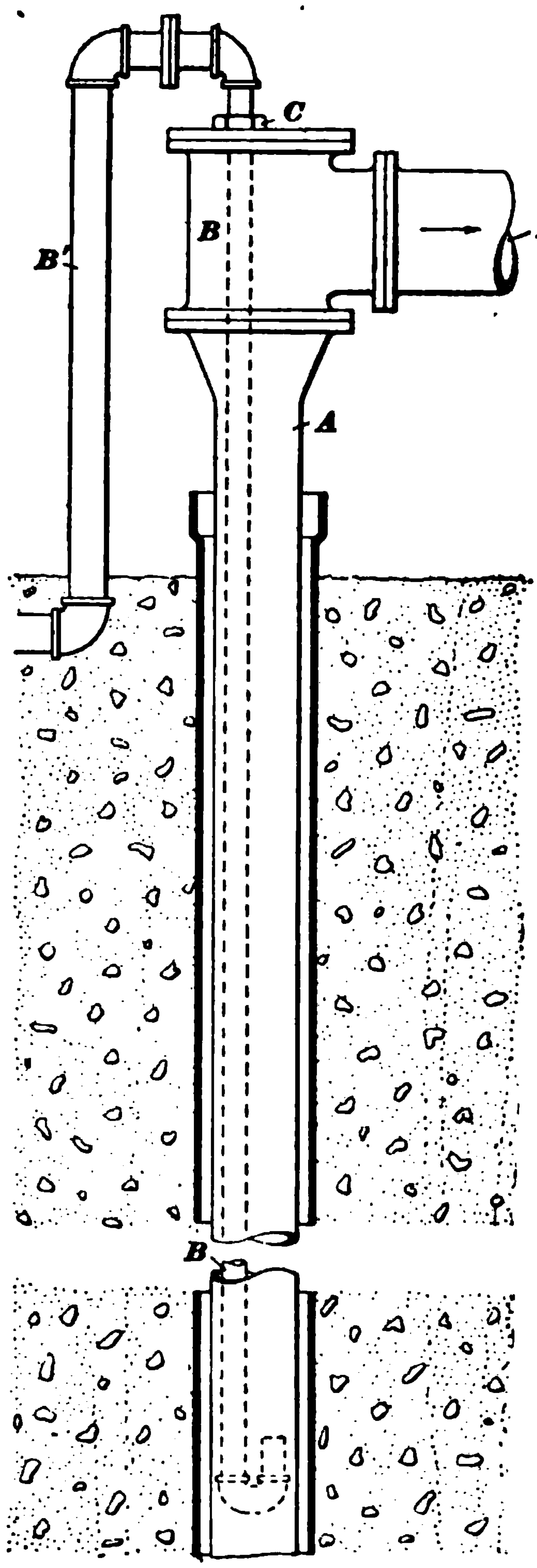


FIG. 14

41. The Air Lift.—In some soils, however, such pits are impracticable, on account of the soft, marshy ground, on account of the presence of quicksand, or on account of large quantities of water in which

the pit must be dug. In such cases, recourse must be had to some form of pump that, when installed on the surface, will pump water from a depth greater than 26 feet. For this purpose, either an *air lift* or a *deep-well pump* must be used.

Fig. 14 shows the main parts of an **air lift.** Inside the well casing, shown by the heavy lines, is lowered a pipe *A*, 1 inch or so smaller in diameter; the lower end of this pipe reaches well below the level of the water to be raised. Inside of this pipe, making an air-tight connection with it at the top, as shown at *C*, is a smaller pipe *B B* that leads from an air compressor through an outside pipe *B'* and reaches nearly to the bottom of the larger pipe *A*. Compressed air is supplied from the compressor and escapes at the lower end of the pipe *B* through the perforations at its bottom. At the beginning of the operation, the water is at the same level on the inside and on the outside of the pipe *A*. When air is forced down the pipe *B* and into the bottom of the pipe *A*, the weight of the water inside the latter pipe is reduced below the weight of the water outside, or in the casing. This difference in weight is due probably to the fact that the contents of the pipe *A* are an intimate mixture of air and water, the air molecules being entangled and intimately associated with the molecules of water. Since water is heavier than air, the water outside the pipe *A* is heavier than this mixture, and, consequently, the water rises in the pipe *A* and flows out at the outlet *A'*. The amount of air added determines the height to which the column of water will rise, although the more air is added, the less water is in the mixture, and the less water is pumped. Fig. 15 shows the pipe in which is the lighter mixture of air and water, the open dots being water.

Fig. 15

The difference in weight between the mixture inside and the water outside may also, especially in small tubes, be due to the fact that the air and the water may form alternate layers in the pipe. This combination, like the other, is

lighter than a column of water of the same height, and therefore the inside column of air and water rises in the tube higher than that on the outside. Fig. 16 shows this condition.

42. The discharge of an air lift can be approximately determined by the following formula:

$$Q = \frac{125\,A}{h}$$

in which Q = gallons of water delivered per minute;

A = cubic feet of air used per minute;

h = depth of water surface, in feet, below point of discharge.

Fig. 16

43. The air lift has the disadvantage that it requires a constant submergence of two-thirds; that is, the delivery pipe must be carried to a depth below the surface of the water equal to twice the distance from the surface of the water to the point of delivery.

The advantages of the air lift are many. There are absolutely no movable parts in it, and therefore it is particularly adapted to the handling of dirty or gritty water, and to the lifting of sewage, mine water, and strong acid or alkali waters that would rapidly corrode the linings of ordinary pumps. For these purposes, the air lift seems to possess ideal cheapness and durability.

44. Deep-Well Pumps.—A deep-well pump is shown in Fig. 17. A cylinder, called a **working barrel** (see W, W, Fig. 18), is placed in the well, reaching down into the water to be raised. This working barrel, which is the most important part of the pump, is a brass cylinder, about 10 feet long, in which two conical valves A and B work. A strainer SS is attached to the lower end of the cylinder; the upper end is threaded to receive the tubing T, or the pipe through which the water is pumped. This tubing is made about $\frac{1}{2}$ inch larger in diameter than the cylinder, in order that

FIG. 17

FIG. 18

the valves may be drawn up for repairing or repacking without disturbing the cylinder or the tubing.

Fig. 19 shows the two **sucker rods**, one of which is hollow, and the other solid; they have practically the same weight. The solid rod works inside of the hollow (see Fig. 18). There are in the working barrel two movable valves *A* and *B*, Fig. 18, similar in all respects except that the upper valve has a connection into which the hollow sucker rod is screwed, and the lower valve is prepared to receive the solid or inside sucker rod. The sucker rods are connected to the crosshead of the engine above ground in such a way that, when one of these rods is on its upward motion, the other rod is on its downward motion. In this way, one of the two valves is

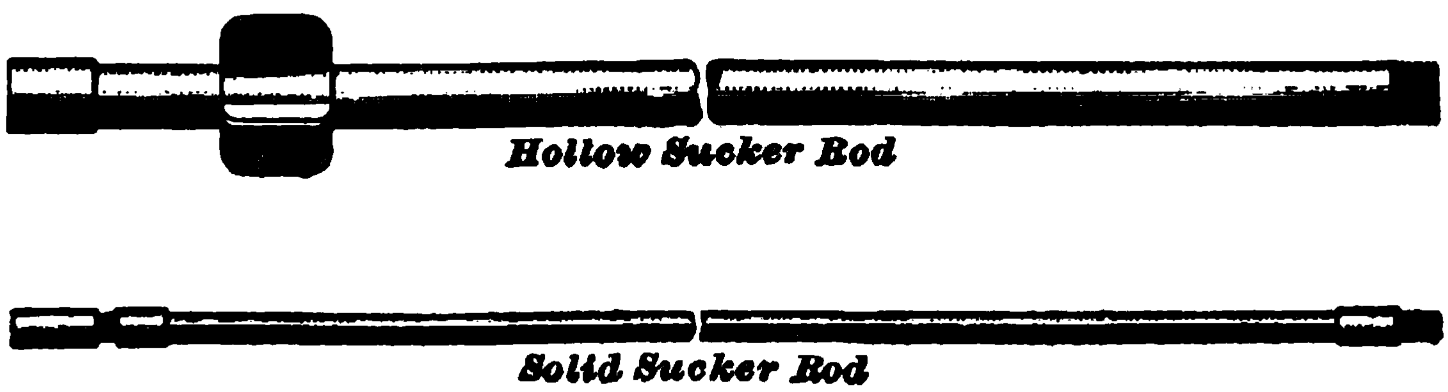


Fig. 19

always moving upwards. In Fig. 18, the valves are represented as being both together, *A* at the top of its stroke and *B* at the bottom of its stroke. As the pump moves, *B* travels upwards and discharges the water above it through the length of its stroke. *A* travels downwards through the water in the tube, its valve allowing the water to pass through to the upper side. When *B* has reached the top of its stroke and stops discharging, *A* is ready to start upwards and discharge in its turn.

It is to be noted that, since the sucker rods act only in tension, the weight of the valves being sufficient to pull them down, they may be flexible, and are therefore usually made of wood, this material being lighter for a long line of rod than steel or iron.

45. Advantages and Disadvantages of a Deep-Well Supply.—The advantages of water derived from driven wells are that it is most likely to be pure, that there is no

expense for land, nor for the construction of reservoirs, and that, generally speaking, the supply can be increased from time to time as needed simply by driving more wells. The disadvantages are a probability of the water being very hard, a possible high temperature, and a great deal of uncertainty as to the quantity of water obtainable and as to the permanence of the supply.

46. Operating Deep Wells.—The facility and economy with which deep wells can be operated depend mainly on the distance from the surface to which water rises naturally. If a battery of wells is used, the elevation of the water is a large factor in the economy of both installation and operation. If the water is within 25 feet of the surface, or at such a reasonable depth that the pumps can be set in a well within that distance of the water, each well of the battery can be connected with a single collecting pipe leading to the pump plungers. If, however, the elevation of the water does not allow this, a deep-well pump must be placed in each well. This greatly complicates the work, and may make another source of supply preferable.

SPRINGS

47. Springs as Sources of Supply.—Of all the sources of water supply, abundant springs, breaking forth at some point where the water may be collected into a basin and thence distributed, are the least frequently used. This is especially true in the United States, where sufficient attention has not been directed to the development of springs as a source of supply. Springs constitute a special case of the flowing well, without the expense of boring. Unlike the deep-driven well, the water from springs is generally of a low temperature. Care, however, must be taken that the spring is not contaminated by surface or subsoil drainage. One of the most noteworthy instances of a spring supply is that of the city of Havana, Cuba, where springs furnish more than 38,000,000 gallons per 24 hours, the water being exceedingly pure and cool. The domestic supply of the city

of Paris, France, is also derived from springs, the water of which is brought a long distance by a magnificent system of aqueducts.

A good spring supply may sometimes be obtained by collecting into one basin a number of small springs. The flow of a spring used as a source of supply must be measured at a time when there is a stage of low water, so as to obtain the minimum yield.

A large supply of spring or ground water may be obtained by driving small tunnels or even by digging trenches in a hillside, and placing in them drain pipes, surrounded by coarse gravel or broken stone. Extensive supplies of water have been obtained in this way, by digging a trench across the flow of an underground stream. In the Arkansas River Valley, California, a trench was dug, the bottom of which was 6 feet below the water level, and water was collected at the rate of 220,000 gallons per day for each 100 feet of trench. On the South Platte River, near Denver, Colorado, there is an excavation 18 feet deep in the water-bearing gravel, which collects water at the rate of $1\frac{1}{4}$ million gallons per day for each 100 feet of trench. Near Hartland, Kansas, an excavation 7,900 feet long supplies water at the rate of 172,000 gallons per day for each 100 feet of trench.

48. Collection of Water From Springs.—Spring water is usually collected in receptacles or chambers made of masonry. If the collecting chamber is built at the end of a tunnel, or if the spring at the surface is merely walled in, care should be taken to prevent pollution of the supply by surface water flowing down the hillside. Protection is secured by making a roof over the collecting chamber and carrying it up the hillside over the spring, and over the area draining directly into the spring, until surface water would be forced to pass through at least 10 feet of soil before reaching the chamber. This cover should preferably be of concrete; a mixture of clay and gravel may, however, be substituted for the concrete if economy is important. The outlet pipe from the chamber should be placed above the bottom, so that there

may be no danger of its being clogged with mud or débris, and it should be provided with a strainer made of brass, in the form of a cylinder, drilled full of $\frac{1}{4}$-inch holes. The combined area of the holes should be at least twice the area of the delivery pipe.

In building a collecting chamber, the spring is first cleaned out and dug down into the impervious stratum and back into the hill, to make room for the collecting chamber. The size of the chamber depends on the number of people to be supplied, and on the flow of the spring. If the latter is sufficient at all times for the demands likely to be made on it, the chamber need be only large enough to supply the delivery pipe. In the crudest form, the chamber, under these conditions, may be a barrel, or a large sewer pipe, set vertically in the ground around the spring. If the flow of the spring is not equal to the demands at all times, there must be some storage. The amount of storage must be determined by measurements of the flow at different seasons, and the demands of the population to be supplied must also be gauged at different times of the day and of the year. Estimates may replace actual measurements in case of necessity, but actual measurements are better. It may be that the demand is but little more than the supply. Then the storage will only need to be from night to day, letting the chamber fill up during the night, the water being used up during the day. It may happen, on the other hand, that during the month of August, for example, the spring does not furnish enough water, even with storage of the night flow. Then, the chamber must be made large enough to store up water and hold it over during the period of drought.

Fig. 20 (from Goodell) shows a collecting chamber *a* with all the necessary accessories. The cross-sectioned parts *p*, *p*, *p* represent the impervious strata by which the spring is formed, and into which the chamber is built; *o* is an opening left in the back wall of the chamber, blocked with large stones to keep out the dirt above, but through which the spring water can enter the chamber *a*. The water in the chamber is held at a constant level by an opening *c* in

the opposite wall, and if the flow into the chamber is greater than the consumption, the water flows through the opening *c* into the bottom of another chamber *b*, and out through a waste pipe *d*. The waste pipe d_1 is used for removing the water in the chamber when the latter is cleaned out, and is ordinarily kept closed by a valve *v*. The

FIG. 20

supply pipe is shown at d_2; it projects some distance into the chamber, and admits the water through a strainer *s*. The entrance into the chamber, ordinarily closed by an iron cover projecting above the surface of the ground to keep out surface water, is shown at *m*. The chamber is ventilated, as such underground structures should always be, by a special vent pipe, or by holes left in the entrance cover.

RIVER AND LAKE INTAKES

49. When water is taken from a river or a lake, the collection is usually very simple. A cast-iron pipe, running out into the water to a permanent depth of from 6 to 10 feet, is often all that is required. River or lake supplies have usually to be pumped, since the water is at the lowest level of the drainage area and therefore the iron collecting pipe will usually be short, running from the river into a pump well that is built on the bank. Where the bottom of the river is muddy, the end of the pipe should be raised above the bottom; this is most simply done by building a small crib around the end of the pipe, weighting the crib with stone, and letting the end of the pipe project above the top of the crib (see Fig. 21). Such a crib should be at least 8 feet square, built of 8″ × 10″ timbers well bolted together, with a bottom well fastened to the cribwork and filled with as many stones as it will carry. The crib is floated to place and then loaded full to hold it permanently. The vertical pipe is left open; it may be above the crib, as shown in the figure, or a few feet below the top of the crib, which is provided with a coarse grating. Railroad rails, spaced 3 or 4 inches in the clear, make a good grating, as they can be bought second-hand and their weight adds to the stability of the crib.

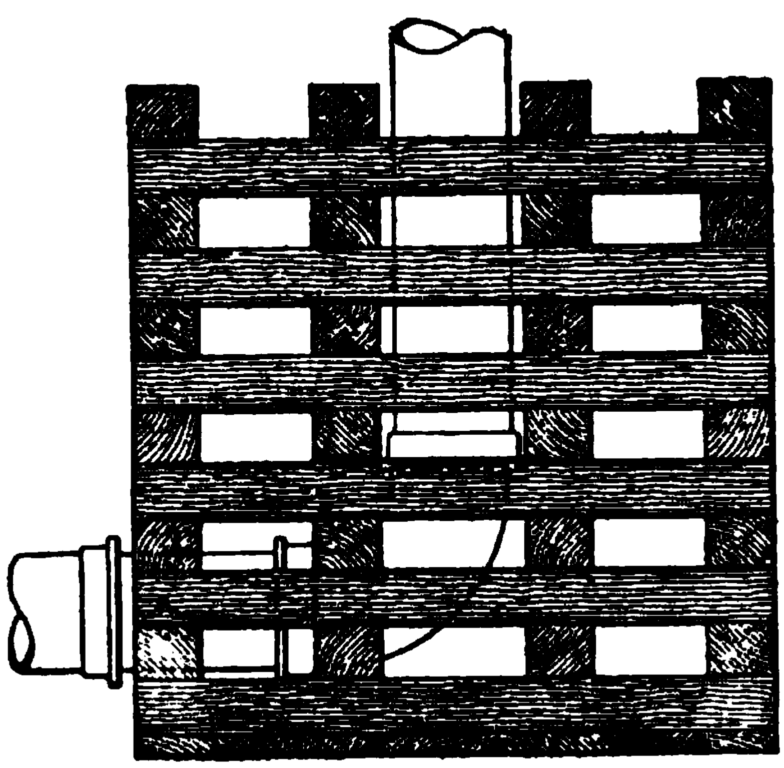


FIG. 21

If the river or lake is subject to violent currents or to severe storms, it is not safe to allow the pipe to rest directly on the bottom, but there should be a trench dredged out of the bottom for protection. This is particularly necessary, notwithstanding the expense, if the bottom is rocky. Piles

may be driven on both sides of the pipe in trench bottoms, and cross-timbers to hold the pipe down may be put in place by divers.

If the intake leads into quiet water of no great variation in level, and with a depth at low water of 6 or 8 feet, the

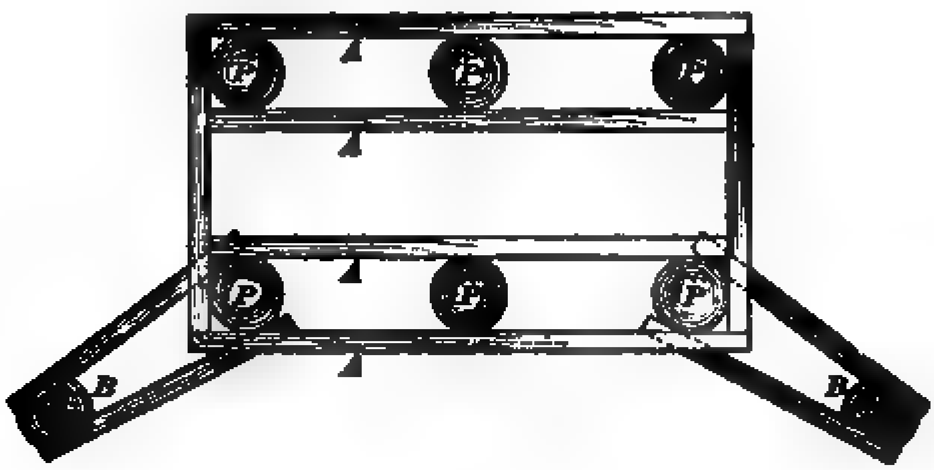


FIG. 22

pipe may end at the bank and be fastened to a support resting on the firm soil, provided there is no danger of washing by the current or the waves. If greater security is required, a small timber-and-pile foundation will be necessary. This may be built, as shown in Fig. 22, by driving eight piles well into the ground, and cutting off their heads level about

3 feet below the bottom of the intake pipe, which is shown at *I* as being just below low water. Six of the piles are driven in two rows, the piles being 3 feet apart each way; $3'' \times 10''$ planks are then spiked on the sides of these piles flush with the top of the latter, as shown at *A*, *A*, *A*, *A*, and also from the piles at the front corners out to the outside piles *B*, *B*, which support the wing or inclined walls. A 3-inch plank floor is then laid over these timbers, on which the masonry can be laid without danger of settlement or wash. The flooring laid out to the outside piles should overhang the timbers so that a floor 30 inches wide can be had for the wall. In order to lay down the timbers and build the wall, it will be necessary to protect the space used by a coffer dam of some sort. For a small wall, as that here described, 2-inch tongued-and-grooved plank, driven close together vertically into the edge of the bank, and, if necessary to keep out the water, doubled so as to break the vertical joints with the first row, will usually be sufficient. A pile of cement bags, filled with sand on the water side of the work, will often answer the purpose.

PRACTICAL APPLICATIONS OF HYDRAULICS

50. Introductory.—The following articles contain some useful applications of the principles and formulas given in *Hydraulics*. Some of the problems treated do not occur very often in practice; however, they do occur occasionally, and the accomplished hydraulician must be prepared to cope with them. These problems are all solved, if solvable, by the formulas already established, but a great deal of ingenuity is often necessary in order to adapt these general formulas to special cases.

51. Approximate Formulas for the Flow of Water in Long Pipes.—In solving problems relating to the flow of water in long pipes, Table IV, given in *Hydraulics*, Part 2, should be used when possible; otherwise, the exact

formulas, if great accuracy is required, must be employed. Where not much accuracy is necessary, the formulas given below are very convenient; they are sufficiently approximate for many purposes. They differ from the exact formulas in that they are based on a constant value of the coefficient f.

The velocity of water flowing in a long pipe is given by the formula

$$v = \sqrt{\frac{2ghd}{fl}}$$

If f is taken equal to .02, which is about its average value, the formula becomes, since $g = 32.16$,

$$v = \sqrt{\frac{64.32\,hd}{.02\,l}} = 56.7\sqrt{\frac{h}{l}d} \qquad (1)$$

If the head per 1,000 feet is denoted by H, the head per foot of length of pipe is $\frac{H}{1{,}000}$; and, since $\frac{h}{l}$ is also the head per foot of length, we have

$$\frac{h}{l} = \frac{H}{1{,}000}$$

Substituting this value in formula **1**,

$$v = 56.7\sqrt{\frac{H}{1{,}000}d} = 1.79\sqrt{Hd} \qquad (2)$$

The general formula for discharge is

$$Q = .7854\,d^2 v$$

or, substituting the value of v from formula **2**,

$$Q = 1.4\sqrt{d^5 H} \qquad (3)$$

Also,

$$H = \frac{Q^2}{1.96\,d^5} = \frac{.5\,Q^2}{d^5} \qquad (4)$$

52. Still more accurate results are obtained from Table XIII, at the end of this Section, in which Q is given in terms of H, and H in terms of Q. This table has been prepared for the most common values of d, using closer values of f than the average value .02.

53. Elevation Above Datum.—In all general hydraulic formulas, the fall or head has been represented by H or h. In engineering operations, heights or elevations are generally stated as above some fixed level plane or

datum, which is usually assumed below the entire work to be executed (see *Leveling*). Near the coast, this datum plane is generally the level of mean low water, and if any subaqueous or any underground work below that level is done, its elevation bears the minus sign, or else the datum is changed so that all elevations will be positive.

If, for example, the elevation of the surface of the water in a certain reservoir is stated as 375 feet, and this reservoir is connected by a pipe with another reservoir whose elevation is stated as 233 feet, the head h to be used in the formulas is 375 − 233, or 142, feet. If the length of the pipe between the two reservoirs is 7,100 feet, the slope s, or head per foot of pipe, is $\frac{142}{7,100}$, or .02, and the head H per 1,000 feet is .02 × 1,000, or 20.

PIPE BRANCHES

54. Pipe With Two Branches.—Let E be the elevation of the surface of the water in a reservoir R, Fig. 23. A pipe P having the diameter d and the length l runs out of the reservoir, and at B is divided into two branches P_1 and P_2, whose diameters and lengths are respectively d_1 and d_2, l_1 and l_2. These two pipes discharge at points whose elevations

FIG. 23

are E_1 and E_2. It is desired to know, when the pipes are discharging freely, how much water is delivered by each of the two branches P_1 and P_2.

Imagine a piezometric tube T to be erected at the point B of embranchment. If the elevation x of the water in this tube when both branches are discharging freely were known, the

problem could be easily solved. It is evident that the quantity discharged by the pipe P under the head $E - x$ at the point of embranchment must be equal to the quantity discharged by the pipe P_1 under the head $x - E_1$, *plus* the quantity discharged by the pipe P_2 under the head $x - E_2$. The general method of procedure for the solution of this problem can best be illustrated by an example.

EXAMPLE.—Referring to Fig. 23, let the data be as follows:

$E = 300$ feet; $l = 3{,}000$ feet; $d = 24$ inches
$E_1 = 250$ feet; $l_1 = 2{,}000$ feet; $d_1 = 18$ inches
$E_2 = 200$ feet; $l_2 = 1{,}500$ feet; $d_2 = 12$ inches

It is required to determine the quantities Q_1 and Q_2 discharged by the pipes P_1 and P_2, respectively.

SOLUTION.—The discharge of P, which is equal to $Q_1 + Q_2$, will be denoted by Q. The head h for the pipe P is $E - x$, or $300 - x$; the heads h_1 and h_2 for P_1 and P_2 are, respectively, $x - 250$ and $x - 200$. Since $\frac{H}{1{,}000} = \frac{h}{l}$ (Art. 50), or $H = 1{,}000 \times \frac{h}{l}$, we have

$$H = 1{,}000 \times \frac{300 - x}{3{,}000} = \frac{300 - x}{3}$$

$$H_1 = 1{,}000 \times \frac{x - 250}{2{,}000} = \frac{x - 250}{2}$$

$$h_2 = 1{,}000 \times \frac{x - 200}{1{,}500} = \frac{x - 200}{1.5}$$

Then (Table XIII),

$$Q = 8.3\sqrt{\frac{300 - x}{3}}$$

$$Q_1 = 3.9\sqrt{\frac{x - 250}{2}}$$

$$Q_2 = 1.4\sqrt{\frac{x - 200}{1.5}}$$

But $Q = Q_1 + Q_2$; hence,

$$8.3\sqrt{\frac{300 - x}{3}} = 3.9\sqrt{\frac{x - 250}{2}} + 1.4\sqrt{\frac{x - 200}{1.5}}$$

Squaring,

$$68.89\left(\frac{300 - x}{3}\right) = 15.21\left(\frac{x - 250}{2}\right) + 1.96\left(\frac{x - 200}{1.5}\right) + 10.92\sqrt{\left(\frac{x - 250}{2}\right)\left(\frac{x - 200}{1.5}\right)}$$

Reducing and solving for x,

$$x = 297.4, \text{ or } 275.4$$

There will frequently be some doubt as to which is the right value of x, so that it will be necessary to try which value will satisfy the

equation $Q = Q_1 + Q_2$. There cannot be two solutions. Trying the second value as the one more likely to be correct, we have,

$$Q = 8.3\sqrt{\frac{300 - 275.4}{3}} = 23.8$$

$$Q_1 = 3.9\sqrt{\frac{275.4 - 250}{2}} = 13.9$$

$$Q_2 = 1.4\sqrt{\frac{275.4 - 200}{1.5}} = 9.9$$

Since $Q_1 + Q_2 = 13.9 + 9.9 = 23.8 = Q$, the selected value of x is correct.

55. Pipe With Several Branches: Numerical Illustration.—A reservoir R, Fig. 24, is situated at an elevation of 500 feet above datum. It is desired to supply three other reservoirs, situated, respectively, at elevations of 310, 330, and 460 feet above datum. The lowest reservoir, whose elevation is 310 feet, is to receive 2 cubic feet per second from a branch pipe 3,500 feet long; the next,

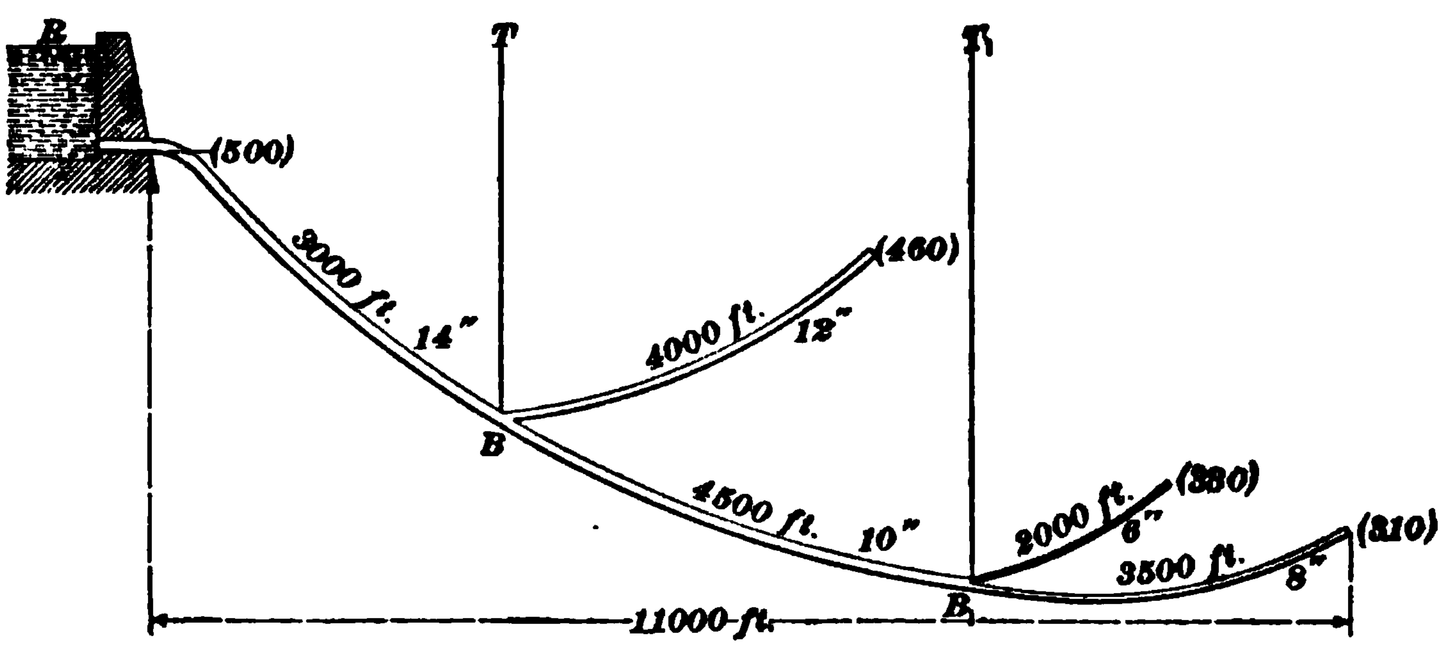


FIG. 24

with an elevation of 330 feet, is to receive 1 cubic foot per second through a branch 2,000 feet long; and the third, with an elevation of 460 feet, is to receive 2.5 cubic feet per second through a branch 4,000 feet long. The first point of embranchment is 3,000 feet from the reservoir, and the second comes 4,500 feet from the first. No pipe is to be less than 6 inches in diameter. All the elevations and lengths of pipe are shown in the figure. It may be noted here that in this figure elevations are placed in

marks of parenthesis. This is a very convenient practice, and should be followed in all drawings where elevations are given.

The size of the pipe to be used will be determined by means of Table XIII. Beginning with the pipe 3,500 feet long, through which 2 cubic feet of water per second is to be delivered at an elevation of 310 feet, we shall make trials of sizes of pipes that will deliver the required amount of water with the head that is available.

Assuming the diameter to be 6 inches, Table XIII gives

$$H = 18.629 \times 2^2 = 74.52$$

which, multiplied by the total length in thousands, or 3.5 (since H is the head in 1,000 feet), gives 260.8 feet as the required head, or the piezometric height at B_1. But this would require the elevation of the water in the piezometric tube to be $310 + 260.8$, or 570.8, which is 70.8 feet higher than the upper reservoir, and manifestly impossible. Therefore, the 6-inch pipe is inadmissible, on account of the great head necessary. An 8-inch pipe will be tried next. The necessary head, found as before, is 59.8 feet, and the piezometric elevation at B_1 becomes $310 + 59.8$, or 369.8, feet. Since the elevation of the reservoir is 500 feet, the head available is probably sufficient. The result also shows the great reduction in head brought about by a comparatively small increase in diameter. This is because the discharge varies with the square root of the head, and with the fifth power of the diameter.

The elevation 369.8 gives a head of $369.8 - 330$, or 39.8, feet over the reservoir whose elevation is 330, and to which 1 cubic foot of water per second is to be delivered through 2,000 feet of pipe. By referring to Table XIII, opposite the 6-inch pipe, it is seen that the head necessary for this delivery is $18.629 \times 1^2 \times 2$, or 37.26, feet. As this is a little less than the available head (39.8), the 6-inch pipe may be used. If the head had been greater than 39.8, a larger pipe would have been required. If the head had been much less than 39.8, a valve would have been used on the pipe to cut down the flow to the desired amount

Before determining the proper diameter of the 4,500-foot pipe lying between the two points of embranchment, it is best to determine the diameter of the 4,000-foot pipe that is to deliver 2.5 cubic feet of water per second at an elevation of 460. Trying an 8-inch pipe in the manner explained above, it is found too small. The next size, a 12-inch pipe, requires, according to the table, a total head of 13.4 at the point of embranchment. This head, added to the elevation of the reservoir, or 460, gives a piezometric elevation of 473.4, which is less than that of the upper reservoir *R*. A 12-inch pipe will therefore be selected.

The pipe lying between the two embranchments must discharge $1 + 2 = 3$ cubic feet per second. The elevation of the upper end has been fixed at 473.4, and that of the lower end at 369.8, and therefore the total head available in this stretch is $473.4 - 369.8 = 103.6$ feet. Therefore, it is necessary to find the diameter of a pipe that will discharge 3 cubic feet at a distance of 4,500 feet, with a head of 103.6 feet, which is equivalent to $\frac{103.6}{4{,}500} \times 1{,}000$, or 23.02, feet per 1,000 feet. It is found from Table XIII that the head *H* corresponding to a 10-inch pipe is

$$1.3674 \times 3^2 = 12.31 \text{ feet}$$

There is, therefore, head enough to deliver more water than the desired 3 cubic feet.

But if the next smaller pipe is tried, it is seen that there is not head enough. Therefore, a 10-inch pipe must be used, and a valve set in to partly throttle the flow. If the pipes are all left open, and an equilibrium of flow is established, the piezometric head would be somewhat greater than 473.4, which would increase the flow through the 6- and 8-inch pipes, and would decrease the flow through the 12-inch pipe.

Finally, it is required to calculate the diameter of the pipe 3,000 feet long that leads from the reservoir to the first point of embranchment. This pipe must carry the total quantity of flow, $2.5 + 1 + 2 = 5.5$ cubic feet per second. The head is $500 - 473.4$, or 26.6, feet, which is equivalent to

$\frac{26.6}{3,000} \times 1,000$, or 8.87, feet per 1,000. It will be found by the table, trying first a 14-inch pipe, that the head required is $.241 \times 5.5^2$, or 7.29, and since there is 8.87 feet available, the 14-inch pipe will be the proper size to adopt.

COMPOUND PIPE LINE

56. Formula for Discharge.—The term **compound pipe line** is applied to a line composed of pipes of different sizes. Such combinations are often found in old systems, and it is important to know how to calculate their discharge. Let a system of pipes P_1, P_2, P_3, P_4, P_5, Fig. 25, of diameters d_1, d_2, d_3, etc., and lengths l_1, l_2, l_3, etc., respectively, lead from a reservoir R, and let h be the total head for the whole compound line. The elevations of the reservoir and outlet

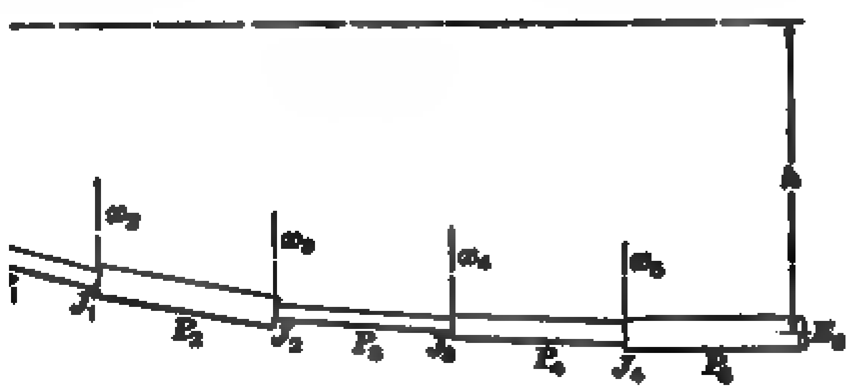


FIG. 25

are, respectively, E and E_5. It is required to determine the discharge Q. Let x_2, x_3, etc. be the piezometric elevations at the junctions J_1, J_2, etc. Then, the value H_1 for the pipe P_1 is $1,000 \times \frac{E - x_2}{l_1}$. Let c_1 be the coefficient by which Q^2 must be multiplied to obtain H_1, as determined from Table XIII. Then,

$$c_1 Q^2 = 1,000 \times \frac{E - x_2}{l_1}, \text{ and } \frac{c_1 l_1 Q^2}{1,000} = E - x_2 \qquad (a)$$

For P_2, the head is $x_2 - x_3$, $H_2 = 1,000 \times \frac{x_2 - x_3}{l_2}$, and, denoting by c_2 the coefficient of Q^2 in Table XIII,

$$c_2 Q^2 = 1,000 \times \frac{x_2 - x_3}{l_2}, \text{ and } \frac{c_2 l_2 Q^2}{1,000} = x_2 - x_3 \qquad (b)$$

Similarly, for the pipes P_3, P_4, and P_5,

$$c_3 Q^2 = 1{,}000 \times \frac{x_3 - x_4}{l_3}, \text{ and } \frac{c_3 l_3 Q^2}{1{,}000} = x_3 - x_4 \qquad (c)$$

$$c_4 Q^2 = 1{,}000 \times \frac{x_4 - x_5}{l_4}, \text{ and } \frac{c_4 l_4 Q^2}{1{,}000} = x_4 - x_5 \qquad (d)$$

$$c_5 Q^2 = 1{,}000 \times \frac{x_5 - E_5}{l_5}, \text{ and } \frac{c_5 l_5 Q^2}{1{,}000} = x_5 - E_5 \qquad (e)$$

Adding equations (a) to (e), and noticing that $E - E_5 = h$,

$$\frac{c_1 l_1 + c_2 l_2 + c_3 l_3 + c_4 l_4 + c_5 l_5}{1{,}000} \times Q^2 = E - E_5 = h;$$

whence $$Q = \sqrt{\frac{1{,}000\, h}{c_1 l_1 + c_2 l_2 + c_3 l_3 + c_4 l_4 + c_5 l_5}}$$

A similar formula applies to any number of pipes.

Example 1.—Fig. 26 represents a reservoir R whose elevation is 500 feet, tapped by a pipe 30 inches in diameter, 4,500 feet long, con-

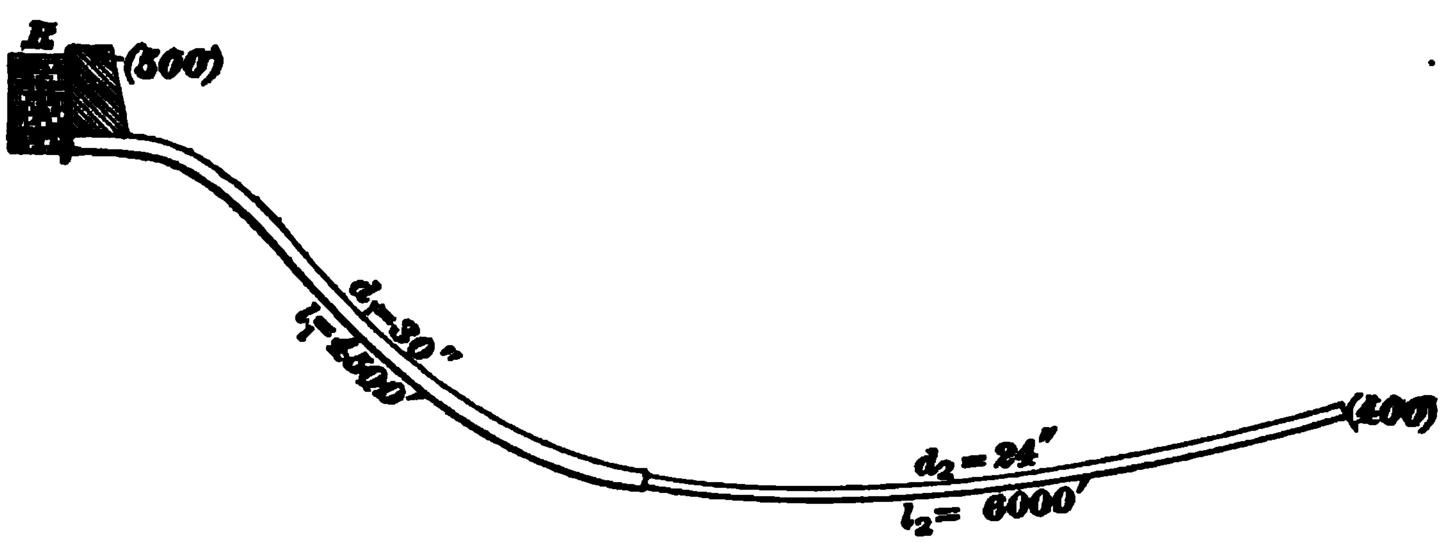


Fig. 26

nected by a reducer with a pipe 24 inches in diameter, 6,000 feet long, discharging freely at an elevation of 400 feet. What is the delivery of the system?

Solution.—Here, $h = 500 - 400 = 100$, $l_1 = 4{,}500$, $c_1 = .00444$, $l_2 = 6{,}000$, and $c_2 = .01446$. Substituting these values in the formula,

$$Q = \sqrt{\frac{1{,}000 \times 100}{.00444 \times 4{,}500 + .01446 \times 6{,}000}} = 30.6 \text{ cu. ft. per sec.} \quad \text{Ans.}$$

Example 2.—To find the discharge of a compound pipe line when the lengths and diameters are as follows: 1,100 feet of 36-inch pipe; 1,500 feet of 24-inch pipe; 1,800 feet of 20-inch pipe; and 2,000 feet of 18-inch pipe. The extremity of the 18-inch pipe is 87 feet below the level of the distributing reservoir.

SOLUTION.—Here, $h = 87$, $c_1 = .00168$, $l_1 = 1{,}100$, $c_2 = .01446$, $l_2 = 1{,}500$, $c_3 = .03769$, $l_3 = 1{,}800$, $c_4 = .06545$, and $l_4 = 2{,}000$. Substituting these values in the formula,

$$Q = \sqrt{\frac{1{,}000 \times 87}{.00168 \times 1{,}100 + .01446 \times 1{,}500 + .03769 \times 1{,}800 + .06545 \times 2{,}000}}$$
$= 19.7$ cu. ft. per sec. Ans.

EXAMPLE FOR PRACTICE

1. (*a*) What is the discharge of a compound pipe line made up of the following lengths and diameters: 2,200 feet of 12-inch; 1,700 feet of 10-inch; 1,100 feet of 8-inch; and 2,000 feet of 6-inch; $h = 120$ feet? (*b*) How much less is the discharge than if the line were all 12-inch pipe? (*c*) How much greater than if the line were all 6-inch pipe?

Ans. $\begin{cases} (a) \text{ 1.62 cu. ft. per sec.} \\ (b) \text{ 4.04 cu. ft. per sec. less} \\ (c) \text{ .66 cu. ft. per sec. greater} \end{cases}$

57. Replacing Compound System by a Single Pipe. It is frequently necessary to replace with a single pipe an old compound system. This can readily be done when the discharging capacity, or Q, and the total head, or h, of the compound system have been determined. The problem is simply to find the diameter of a pipe when the length, head, and discharge are given, and can be solved by Table IV in *Hydraulics*, Part 2, or by Table XIII, Art. **51**, in the manner illustrated in the following example.

EXAMPLE.—To replace with a single pipe a compound pipe 5,000 feet long, the head being 65 feet, and the discharge, 10 cubic feet per second.

SOLUTION.—Here, $H = 1{,}000 \times \frac{65}{5{,}000} = 13$. Denoting by c the coefficient of Q^2 in Table XIII, we may write $H = c\,Q^2$, and, therefore,

$$c = \frac{H}{Q^2} = \frac{13}{10^2} = .13$$

Looking in the table, the coefficient of Q^2 for a 16-in. pipe is seen to be .12, the nearest value to .13. These two values are practically equal, and a 16-in. pipe may be considered to answer the requirements.

EXAMPLES FOR PRACTICE

1. Find the diameter of a pipe necessary to replace a compound pipe line having the following elements: 2,700 feet of 16-inch pipe; 1,300 feet of 12-inch pipe; and 1,250 feet of 10-inch pipe. The point of discharge is 107 feet below the level of the reservoir. Ans. 12 in.

2. The discharge through a compound pipe line 4,900 feet long is 8 cubic feet per second. The point of discharge is 80 feet below the level of the distributing reservoir. What must be the diameter of a single pipe that will replace the old compound system? Ans. 14 in.

ROUGHNESS OF PIPES

58. Effect of Rough Pipe on Formulas.—As explained in *Hydraulics*, a **smooth pipe** is a pipe presenting a clean surface of absolutely uniform diameter, that is, having no depressions or elevations on its interior surface. A **rough pipe** is a pipe that, from contact with certain kinds of water, has become roughened by the development of small vegetable growths, or by other accumulated deposits or alterations, such as rust, or that has been jointed in a way to form small projections or depressions at the joints. These conditions not only cause a decrease in the diameter of the pipe, but have a very considerable effect on the velocity, as is shown by the values of f in different experiments with pipes of varying smoothness. In designing a pipe line, care should be taken to use a proper value for the coefficient f. It was stated in *Hydraulics* that it is wise and safe to compute the diameter of a pipe for a condition of absolute roughness, making f twice what it is for smooth pipes.

59. Tuberculation.—By far the most serious cause of roughness in pipes is tuberculation. By **tuberculation** is meant a system of conical projections, or **tubercles,** forming on the interior surface of a pipe, originally from some rust spot, and varying in height from $\frac{1}{8}$ to 1 inch, with irregular and varying bases. These tubercles are usually scattered at frequent intervals over the surface, although they often coalesce and form what resembles a large rough blister. They are formed only in pipes that are not properly coated, and even then not always, since not all waters produce them. These tubercles are not solid, and are easily removable by the thumb nail; yet their influence on the discharging capacity of pipes is very great. A rough pipe 4 inches in diameter, badly tuberculated, may have but 12 per cent. of its original

capacity. A 48-inch pipe in the same condition may have its capacity reduced 25 per cent. As tubercles rarely attain a height of more than 1 inch, the proportion of reduced area in a 4-inch pipe is very much greater than in one 48 inches in diameter.

60. Pipe Coating.—The subject of pipe coating is one that rarely receives from engineers the attention it deserves. Much depends on the quality of the coating as well as on the thoroughness with which it is applied. Mr. Rafter, an eminent hydraulic engineer, states, as a result of his wide experience, that many American pipe foundries apply no coating whatever, and that others apply the coating so carelessly as to make its subsequent failure fairly certain. In some cases, the formula for the mixture is not known to the proprietors, cheap, patented preparations being bought in the open market. His impression seems to be that the foundrymen consider the subject of pipe coating of small importance, and that, so long as the preservative adheres and hardens readily, every necessary condition is deemed to be fulfilled. The fact that the material used for producing a proper preservative is a complex matter and subject to special conditions to secure the best results seems to have been overlooked. He illustrates this point by the statement that water mains, properly coated by Dr. Angus Smith's protective coal-tar varnish, were about as perfect and free from blemish in 1906 as when laid in 1873. Experience in many cities confirms the views of this eminent engineer.

This Angus Smith coating is not a patented article, and, if insisted on, will be furnished and used by any large foundry at an additional expense of but a few cents per ton. The process is simply to immerse the pipe while hot in a bath of boiling coal pitch, from which all naphtha compounds have been removed. The residuum is hard, insoluble, odorless, and tasteless. This coating, if applied over any rust spots, will subsequently peel off; therefore, it is necessary that the pipe be dipped as soon as it leaves the molds and while hot, so that its pores may absorb as much of the mixture as

possible. Mr. Rafter concludes: "The maintenance of the integrity of the coating is a matter of supreme importance, and hair-splitting formulas are of absolutely no use, so long as an indefinite reduction of delivery is possible, due to a more or less constant deterioration of the interior coating. As regards cast-iron water mains, the coal-pitch preparation of Dr. Angus Smith as originally applied at Manchester is the best thus far devised. So far as is definitely known, it protects the pipe indefinitely, if applied strictly in accordance with the original specification."

It would thus appear that the engineer not only controls to a large extent the length of life of a pipe system, but may materially diminish the first cost by insisting on a coating that he is confident will preserve the full diameter of the pipe and thus allow the use of a smaller pipe than would be necessary if the pipe were expected to become tuberculated.

TABLE I

WATER CONSUMPTION, FOR 1895, IN BOSTON AND SUBURBS

Character of House	Water Consumption Gallons per Day	
	Per Family	Per Head
Highest-cost apartment house	221	59
First-class apartment house	185	46
Moderate-class apartment house	123	32
Poorest-class apartment house	80	17
Average of all apartment houses	139	36
Average of metered houses	221	44
Best-class houses	118	23
Middle-class houses	80	20
Moderate-cost houses	95	20
Low-cost houses	55	12
Modern houses	132	26
Houses with one faucet	35	7

TABLE II

WATER CONSUMPTION ACCORDING TO POPULATION

Population	Number of Cities Included	Consumption per Head per Day Gallons
10,000 to 20,000	26	112
20,000 to 30,000	17	115
30,000 to 40,000	9	120
40,000 to 50,000	8	130
50,000 to 60,000	6	127
60,000 to 70,000	5	135
70,000 to 80,000	5	120
80,000 to 90,000	4	107
90,000 to 250,000	6	153
250,000 to 500,000	3	212

TABLE III

WATER CONSUMPTION AS AFFECTED BY METERS

Percentage of Taps Metered	Consumption per Day per Head Gallons
Less than 10 per cent.	153
Between 10 and 25 per cent.	110
Between 25 and 50 per cent.	104
More than 50 per cent.	62

TABLE IV

WATER CONSUMPTION FOR VARIOUS PURPOSES

Purpose for Which Used	Consumption in Gallons per Head per Day		
	Minimum	Maximum	Average
Domestic	15	40	25
Manufacturing	5	35	20
Public	3	10	5
Waste	15	30	25
Total	38	115	75

TABLE V

WATER REQUIRED FOR FIRE

Population	Number of Fire Streams Required Simultaneously According to			
	Freeman	Shedd	Fanning	Kuichling
1,000	2 to 3			3
4,000			7	6
5,000	4 to 8	5		6
10,000	6 to 12	7	10	9
20,000	8 to 15	10		12
40,000	12 to 18	14		18
50,000			14	20
60,000	15 to 22	17		22
100,000	20 to 30	22	18	23
150,000			25	34
180,000		30		38
200,000	30 to 50			40
250,000				44
300,000				48

TABLE VI

DAILY WATER CONSUMPTION, FOR 1897, IN DIFFERENT CITIES

City	Gallons per Head per Day	City	Gallons per Head per Day
Allegheny, Pa. . .	247	Natchez, Miss. . .	27
Atlanta, Ga. . . .	42	New Orleans, La. .	37
Boston, Mass. . .	100	New York, N. Y. .	116
Buffalo, N. Y. . .	271	Philadelphia, Pa. .	215
Camden, N. J. . .	200	Portsmouth, N. H.	30
Charleston, S. C. .	27	Providence, R. I. .	57
Chicago, Ill. . . .	109	Rochester, N. Y. .	71
Cincinnati, Ohio .	135	St. Paul, Minn. . .	60
Cleveland, Ohio .	142	San Francisco, Cal.	63
Dayton, Ohio . . .	50	Washington, D. C.	200
Fall River, Mass. .	43	Yonkers, N. Y. . .	200

TABLE VII

DAILY AVERAGE CONSUMPTION FOR VARIOUS MONTHS, IN TERMS OF DAILY AVERAGE FOR YEAR

Month	Daily Consumption Per Cent.	Month	Daily Consumption Per Cent.
January . . .	87.2	July	123.0
February . . .	89.0	August . . .	113.5
March	88.6	September . .	109.4
April	89.7	October . . .	103.0
May	99.8	November . .	92.1
June	114.0	December . .	88.7

TABLE VIII

RATIO OF MAXIMUM TO AVERAGE CONSUMPTION

City	Ratio of Monthly Maximum to Average Consumption	Ratio of Daily Maximum to Average Consumption
Boston, Mass.	114	119
Buffalo, N. Y.		168
Chicago, Ill.	108	116
Cincinnati, Ohio	124	153
Cleveland, Ohio	114	146
Columbus, Ohio	107	157
Dayton, Ohio	118	178
Detroit, Mich.	117	150
Fall River, Mass.	115	
Louisville, Ky.	127	135
Marquette, Mich.	139	194
Milwaukee, Wis.	113	
Newton, Mass.	125	143
Pawtucket, R. I.	111	153
Philadelphia, Pa.	110	122
Woonsocket, R. I.	122	155

TABLE IX

WATER REQUIRED FOR DOMESTIC AND FIRE PURPOSES, FOR POPULATIONS UP TO 200,000

Population	Water Required Gallons per Minute			Water Required Cubic Feet per Second			Water Required Gallons per Day			Number of Simultaneous Fire Streams Necessary
	Domestic	Fires	Total	Domestic	Fires	Total	Domestic	Fires	Total	
1,000	42	700	742	.1	1.6	1.7	60,480	1,008,000	1,068,480	3
2,000	92	990	1,082	.2	2.2	2.4	132,480	1,425,600	1,558,080	4
3,000	146	1,212	1,358	.3	2.7	3.0	210,240	1,745,280	1,955,520	5
4,000	202	1,400	1,602	.5	3.1	3.6	290,880	2,016,000	2,306,880	6
5,000	260	1,565	1,825	.6	3.5	4.0	374,400	2,253,600	2,628,000	6
6,000	321	1,715	2,036	.7	3.8	4.5	462,240	2,469,600	2,931,840	7
7,000	383	1,852	2,235	.9	4.1	5.0	551,520	2,666,880	3,218,400	8
8,000	446	1,980	2,426	1.0	4.4	5.4	642,240	2,851,200	3,493,440	8
9,000	510	2,100	2,610	1.1	4.7	5.8	734,400	3,024,000	3,758,400	9
10,000	575	2,214	2,789	1.3	4.9	6.2	828,000	3,188,160	4,016,160	9
20,000	1,267	3,130	4,397	2.8	7.0	9.8	1,824,480	4,507,200	6,331,680	13
40,000	2,793	4,427	7,220	6.2	9.9	16.1	4,021,920	6,374,880	10,396,800	18
50,000	3,602	4,950	8,552	8.0	11.0	19.1	5,186,880	7,128,000	12,314,880	20
60,000	4,434	5,422	9,856	9.9	12.0	22.0	6,384,960	7,807,680	14,192,640	22
100,000	7,938	7,000	14,938	17.7	15.6	33.3	11,430,720	10,080,000	21,510,720	28
150,000	12,603	8,573	21,176	28.1	19.1	47.2	18,148,320	12,345,120	30,493,440	35
180,000	15,514	9,392	24,906	34.6	20.9	55.5	22,340,160	13,524,480	35,864,640	38
200,000	17,494	9,900	27,393	39.0	22.1	61.0	25,191,360	14,256,000	39,445,920	40

TABLE X

MONTHLY AND ANNUAL RAINFALL FOR FOUR AMERICAN CITIES

City		Duration of Period Years	Monthly Average												Annual Average
			Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	
Boston . .	Mean	74	3.98	3.78	4.36	4.06	3.79	3.27	3.71	4.39	3.55	3.84	4.31	3.96	47.00
	Max.		8.84	9.98	11.75	10.83	10.38	8.01	12.38	12.10	11.95	8.78	11.63	9.04	67.72
	Min.		.35	.80	.96	.20	.25	.30	.85	.34	.23	.80	.81	.26	27.20
Philadelphia	Mean	68	3.45	3.08	3.53	3.43	3.78	3.86	4.12	4.50	3.54	3.28	3.39	3.41	43.35
	Max.		7.84	6.64	6.71	9.76	7.42	10.95	11.81	16.84	12.09	10.05	7.96	6.26	61.29
	Min.		.73	.82	.69	.59	.19	.74	.96	.62	.20	.30	.80	.26	29.57
Charleston	Mean	11	3.50	3.88	3.69	3.27	4.17	3.98	3.44	4.78	3.16	2.63	2.84	2.73	42.07
	Max.		5.15	8.10	8.94	4.99	6.94	7.74	6.31	9.05	6.29	6.30	5.37	5.40	59.23
	Min.		1.88	2.08	1.57	1.97	2.08	1.40	.42	1.56	1.16	.59	1.16	1.60	32.82
Denver . .	Mean	19	.58	.49	.88	2.07	2.77	1.38	1.67	1.54	.81	.81	.68	.63	14.30
	Max.		2.35	1.22	2.36	4.94	8.57	4.98	4.32	2.68	2.89	2.15	1.93	2.32	20.12
	Min.		.10	.11	.20	.05	.09	.09	.33	.33	.02	.12	.08	.04	9.33

TABLE XI

MONTHLY EVAPORATION, IN VERTICAL INCHES, FOR BOSTON AND ROCHESTER

Month	City	
	Boston, Mass.	Rochester, N. Y.
January	.96	.52
February	1.05	.54
March	1.70	1.33
April	2.97	2.62
May	4.46	3.93
June	5.54	4.94
July	5.98	5.47
August	5.50	5.30
September	4.12	4.15
October	3.16	3.16
November	2.25	1.45
December	1.51	1.13

TABLE XII

YIELD OF WATERSHED

Drainage Areas of From 20 to 200 Square Miles

Name of Stream	Drainage Area Square Miles	Minimum Flow Gallons per Day per Square Mile
Sudbury	78	234,000
Hale's Brook	24	877,500
Croton (west branch)	20	130,000
Ramapo	160	910,000
Pequannock	63	845,000
Paulinskill	126	845,000
Pequest	83	1,105,000
Tohickon	102	6,500
Neshaminy	139	58,500
Perkiomen	152	325,000
Rock Creek	64	741,000
Hackensack	115	1,235,000
Drainage Area of From 200 to 2,000 Square Miles		
Concord	361	1,105,000
Charles	215	1,300,000
Housatonic	790	1,072,500
Croton	339	975,000
Passaic	797	1,105,000
Schuylkill	1,800	1,105,000
Raritan	879	910,000
Potomac	920	143,000
Greenbrier	870	780,000
Shenandoah	770	1,085,500
Neuse	1,000	1,254,500
Great Egg Harbor	216	1,755,000

TABLE XIII

APPROXIMATE FORMULAS FOR FLOW THROUGH SMOOTH CAST-IRON PIPES

Size of Pipe	Value of f	H In Terms of Q	Q In Terms of H
4	.024043	$147.25\ Q^2$	$.08241 \sqrt{H}$
6	.023097	$18.629\ Q^2$	$.23169 \sqrt{H}$
8	.022308	$4.2696\ Q^2$	$.48395 \sqrt{H}$
10	.021803	$1.3674\ Q^2$	$.85517 \sqrt{H}$
12	.021257	$.53576\ Q^2$	$1.3662 \sqrt{H}$
14	.020692	$.24129\ Q^2$	$2.0358 \sqrt{H}$
16	.020193	$.12078\ Q^2$	$2.8775 \sqrt{H}$
18	.019719	$.06545\ Q^2$	$3.9089 \sqrt{H}$
20	.019229	$.03769\ Q^2$	$5.1512 \sqrt{H}$
24	.018359	$.01446\ Q^2$	$8.3160 \sqrt{H}$
30	.017215	$.00444\ Q^2$	$15.0024 \sqrt{H}$
36	.016217	$.00168\ Q^2$	$24.3827 \sqrt{H}$
42	.015313	$.00073\ Q^2$	$36.8896 \sqrt{H}$
48	.014488	$.00036\ Q^2$	$52.9553 \sqrt{H}$

WATER SUPPLY

(PART 2)

PIPE LINES

METHODS OF DISTRIBUTION

1. Gravity System.—Systems of water distribution may be divided into three classes: *gravity*, *distributing-reservoir*, and *direct-pressure* systems. In the **gravity system**, the source of the water, whatever it may be, is at such an elevation, with respect to the city or town supplied by the system, that full and adequate pressure in the pipes is obtained directly from that elevation. This is by far the best of the three systems, since it insures a pressure in the pipes at all times; the amount of water available is ample, unless the supply fails altogether; and when once the system is installed, the expense of distribution is nothing. This last advantage is of great value, as can be seen from the following considerations: The cost of pumping is about 5 cents per million gallons lifted 1 foot. A town of 10,000 people uses about 1,000,000 gallons a day, and, to secure good pressure, that amount of water must be lifted at least 150 feet. The cost will then be about $7.50 a day, or $2,737.50 a year, which is the interest, at 5 per cent., on $54,750. Therefore, a city or town will be justified in spending this amount for a gravity supply in order to avoid pumping. To furnish and lay a main for a small city costs about $5 a foot, so that the amount just named will allow a main about 2 miles long to be built to replace a pumping system.

The money available for pipe lines, however, is affected by the amount necessary to construct such head-works as dams, intakes, etc.

The gravity system has the additional advantage, not to be estimated in dollars and cents, of a certainty of water at all times, not affected by the condition of the pumps, the shortage of fuel supply, the faithfulness of employes, nor many other factors that make pumping uncertain.

2. **Distributing-Reservoir System.**—When it is not practicable to obtain a source of supply sufficiently high to permit a gravity system, pumps must be used. The **distributing-reservoir system** includes the source of supply, such as a lake, a river, or wells; pumps by which the water is lifted; and a reservoir or standpipe, or merely a tank into which the water is forced, and from which it runs to the city by gravity. If a reservoir is constructed at a suitable elevation, the result, so far as a satisfactory distribution is concerned, is equal to the gravity supply, and the system is inferior only in the constant outlay for running the pumps. The reservoir is made large enough to hold a day's supply, so that any accident to the pumps may be repaired without a water famine, and the pumps need not be run at night, which is a saving in expense. If water can be obtained to supply power, the cost of running is much reduced, and the plant becomes independent of the fuel supply, which is uncertain. Where land at a suitable elevation cannot be obtained, a standpipe or an elevated tank is used instead of a reservoir. The difference between the operation of a standpipe or tank and that of an ordinary reservoir is that, in the latter, water is pumped into the reservoir through a supply main, and is distributed to the city through a separate system of pipes; while, in the standpipe and tank, the water is usually pumped into the distributing main itself, which is a continuation of the street main, and leads to the standpipe or tank, into which it opens.

The amount of storage in a standpipe or tank is small, probably not more than 100,000 gallons (a few hours' supply

only), and hence there is always some danger of the supply becoming insufficient at times when the pumps need repairs. A standpipe or tank, however, has one important advantage over an ordinary reservoir; namely, that it allows the water level to be maintained at the proper height for a moderate pressure, sufficient for all domestic needs, and when, as in times of fire, the pressure needs to be increased, a valve can be closed at the base of the standpipe or tank, and the pump pressure made as high as the pump can give or the mains resist. In the gravity system, on the other hand, if a pressure sufficient for fires is provided, the stress in the pipes is excessive at all other times, and causes leakage and breaks. The design and construction of reservoirs, standpipes, and elevated tanks is treated in another Section.

3. **Direct-Pressure System.**—In the **direct-pressure system,** water is forced by the pumps directly into the distributing mains, there being no provision for storage. The quantity and the pressure of the water supplied are regulated by the working of the pumps. Some years ago, this system was popular for cities built on flat level areas, saving, as it did, the cost of the standpipe or tank; and the pumps used (the Holley pumps, from which the system is often called the *Holley system*) were supposed to be so designed as to be readily adapted to differences in pressure and delivery. This system is now, however, used only under exceptional conditions, as it is neither convenient, certain, nor economical.

4. **Pressure in Water-Supply Pipes.**—The pressure of water in pipes is usually indicated by an ordinary steam or pressure gauge attached to the water pipe at some point. For the proper supply of the upper stories of residences, a pressure, *at the street level*, of from 25 to 30 pounds per square inch is considered sufficient; in business districts, with their high office buildings, from 35 to 40 pounds is necessary.

5. Fire service requires a pressure greater than that provided for domestic use, both because the rate at which water is used is greater than in domestic consumption, and because the height to which the water must be thrown is

greater. A pressure of 80 to 100 pounds is not too great when the pipe pressure is depended on for forcing the water through hose to a fire. It is often cheaper, however, especially in large cities, to provide a lower pressure for ordinary occasions, and use fire-engines in case of fire. Like many other engineering questions, the problem of pressure requires for its solution a careful weighing of different conditions and possibilities, on which a comparison of costs can be made. There is a definite and certain value in a fire pressure ready at all times of day and night, a pressure that makes it possible to attach hose to a hydrant at any time and obtain a powerful stream of water. On the other hand, such a pressure means heavier pipe in the street, and often a greater expense in the construction of high reservoirs or standpipes, a greater expense for pumping to a height unnecessary during a large part of the time, and continual expense in repairing leaks in house fixtures.

A better plan is to have a reservoir that supplies the necessary pressure at ordinary times, and, by means of a cut-off valve, allows the pumps to speed up and increase the pressure in the mains alone at times of fire. If automatic and reliable devices are installed, by which this change of pressure can be made certain at critical times, such an arrangement is nearly ideal. Additional pumps may be installed, allowing the regular pumps to run at full capacity, in which case carefully banked fires must be maintained under the auxiliary boilers at all times. Or, the auxiliary pump may be run by an electric motor that can be started at any time.

6. No installation that includes a pump should be without an extra pump for cases of emergency. All pumps need occasional repairing, and the service should not be interrupted while repairs are being made. Furthermore, a breakdown is most probable when the demands are heaviest and when a diminution or interruption of the supply would be most serious. It may be stated, as a fundamental rule, that every piece of apparatus that may, by continual use, get out of order and thereby jeopardize the supply, should be in duplicate.

DESIGN OF A PIPE SYSTEM

7. Surveys and Maps.—In designing a water supply for a town, a complete survey of the town should be made, and a map prepared showing all the present and the proposed streets. A full set of levels should be taken, determining the elevations of all street intersections, of the source of supply, of all points where the grade changes, and of any other points whose elevations may affect the design. The map should show the depth and width of all rivers, streams, and gulleys, and all other physical features that may be of value for planning or constructing the system. Elevations are most conveniently shown by contours, although those of important points should be indicated by numbers. The density of population should be ascertained for the different sections of the town, that is, the average number of persons per acre in each district. This density will vary from about 15 per acre, in regions of scattered residences, to about 90 per acre, in tenement-house districts. This difference in density can be conveniently shown on the map by light tints of different colors, or by increasing the depth of tint to show increasing density of population.

The map serves both as an aid in laying out the distribution system and as a record of the system when installed. It is convenient to have a large map, 3 or 4 feet square, on which the entire system can be shown and studied as a whole; and also a series of maps about 24 inches square, on which sections of the city can be drawn to a large scale (80 feet to 1 inch is a suitable scale). On the large map are shown the positions of the reservoir or pumping station, and of the mains, with all the laterals, cross-pipes, blow-offs, dead ends, etc.; on the small maps are shown the exact positions of the pipes in the streets, with the hydrants and valves and all other appurtenances. The large-scale maps should be drawn with sufficient accuracy to show by the frontages of the several properties the approximate assessments to be made, if that method of collecting money for construction is adopted.

8. Location and Size of Pipes.—In general, the water should go from the reservoir or other source to the center of population as directly as possible. The application of this principle, however, is very difficult, and the engineer, in determining the location, must make a very careful study of all the conditions involved. Where the population is distributed uniformly over a large level area, the principle does not apply, and the distribution is usually made in the manner indicated in Fig. 1, which shows the main *A B* leading from the reservoir, and submains leading from both sides of the main to cover the entire area. By this arrangement, the

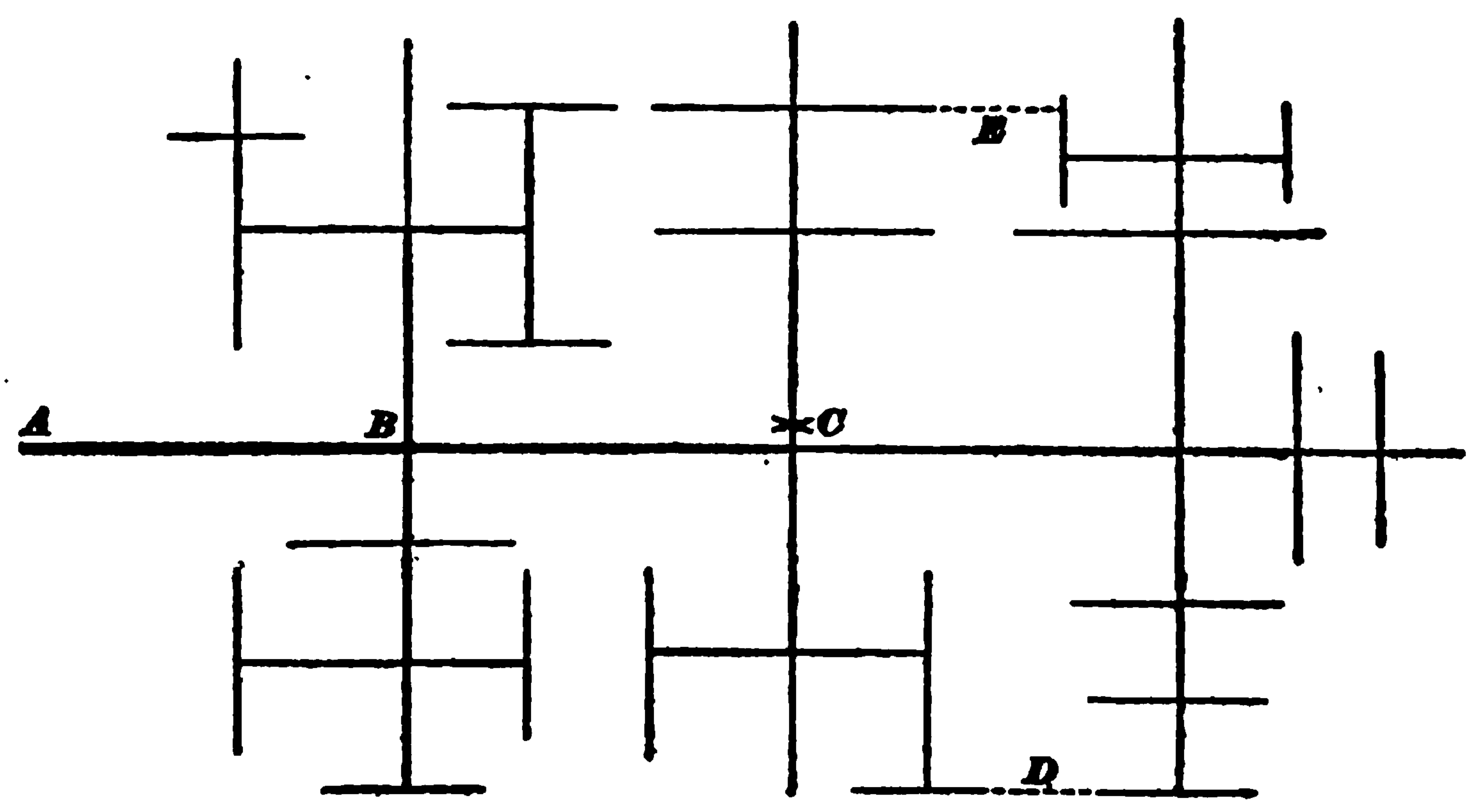


FIG. 1

quantity of water needed for each section of the city is estimated, and the size of the pipe determined for that quantity with the head available. The arrangement has the further advantage that, in case of a break, one valve, as at *C*, can be closed, and a section of the system cut off. But, although pipes are frequently designed according to this system, it has the disadvantage that, at the end of each pipe, there are so-called **dead ends**, where the water becomes stagnant and sediment accumulates.

9. It is found more satisfactory to connect all the dead ends, where possible, as shown at *D* and *E*, Fig. 1, so that water may circulate around the system, and a hydrant or

faucet at D or E may draw water from either submain. In cities that are laid out on the rectangular plan, the connections of dead ends form a regular gridiron system in which the submains run in one direction at intervals of about four blocks, and connect with small pipes at the extreme ends and at street intersections. This arrangement is shown in Fig. 2. In this system, no stagnation is possible, since the water is continually circulating. It is evident, however, that in this case there is no possibility of accurately computing the sizes of the several pipes. When a faucet is opened, the pressure at once drops in all the connecting pipes. But if the assumption is made that any section, as that enclosed by the dotted lines, is supplied entirely by the submain leading to that section, the assumption is on the side of safety, since any point of that section will also receive water from the submains adjoining on each side. So, in calculating the sizes of mains, it is assumed that the total quantity needed in any district is to be delivered at the extremity of the main supplying that district.

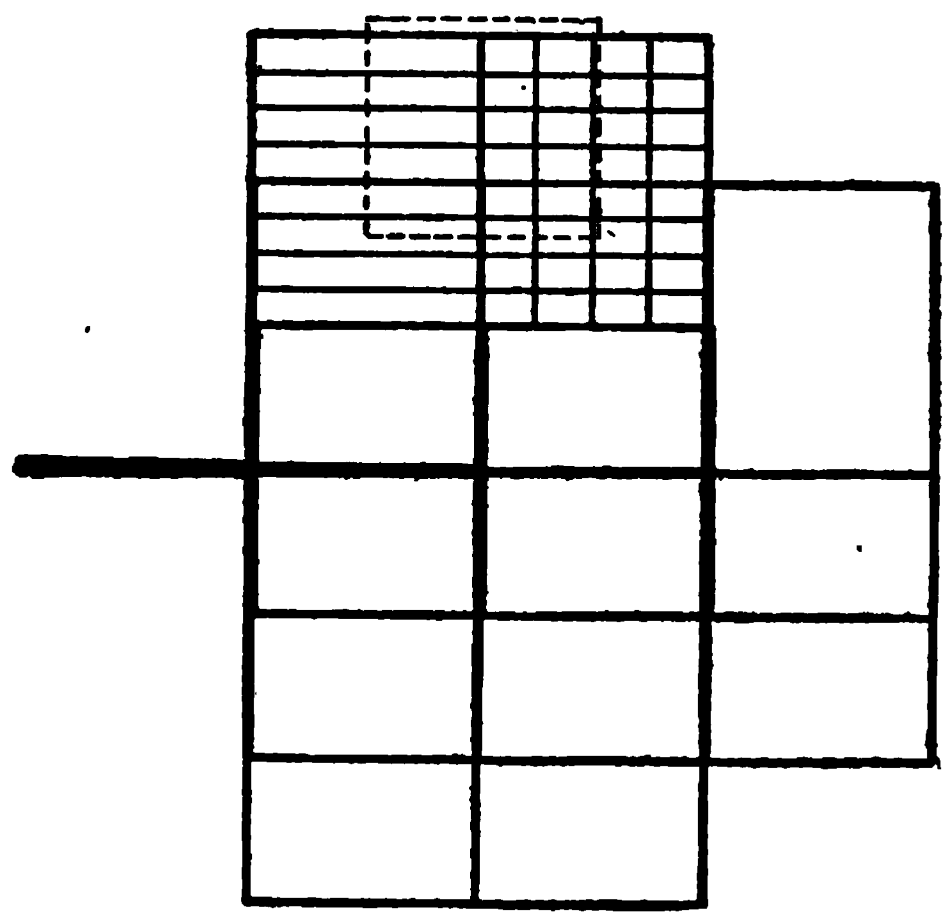


FIG. 2

10. The gridiron system has the disadvantage that, while valves may be so placed that any block can be cut out by closing two valves, one at each end of the block, a large number of valves are needed, four for each intersection or cross. In practice, a system is often laid out that is a compromise between the gridiron and the dead-end system, connections being provided where possible without largely increasing the cost of installation; where there is

no convenient way of connecting dead ends, they are left unconnected, and the sediment is blown out occasionally through hydrants or specially provided blow-offs.

11. The size of laterals is largely a matter of expediency. A 4-inch pipe will carry water enough for the domestic uses of about 1,000 persons, or, with the usual density of suburban residences, for about 1 mile of street. But for fire purposes, a 4-inch pipe will deliver water at a rate barely sufficient to supply one fire stream. If, therefore, the laterals are to furnish fire protection, a 4-inch pipe should not be used. For one block that can be reached from hydrants at the end fed by large pipes, a 4-inch pipe will be ample for domestic needs alone. The difference in cost between a 4-inch and a 6-inch pipe is only about 18 cents per lineal foot, and it is generally believed to be poor judgment to save that amount at the expense of the fire protection given by the larger pipe. Better fire protection is afforded by an 8-inch pipe than by a 6-inch pipe, and if the lateral is in a street where large buildings stand or where serious fires are probable, an 8-inch pipe should be used.

12. A common way of arranging the pipes is to lay the mains and submains so that they will not be farther apart than four blocks in one direction and eight in the other—to run 8- or 10-inch pipes every four or five blocks, and make the pipes at the intermediate blocks 6 inches. Some cities lay 6-inch pipe lengthwise of the blocks and 8-inch pipe across the blocks.

13. It will be generally found that, in small towns, if the mains and street pipes are proportioned to meet the requirements necessary for fire extinguishment, they will be more than sufficiently large for all other public and private needs; while, in large cities, say of over 100,000 inhabitants, the reverse conditions obtain. No general rule can be laid down for the proper dimensions of the submains and street pipes; in each case, the probable fire and domestic requirements for each street must be determined, and the piping proportioned accordingly. It is always better to have the pipes too large

than too small, but it is useless waste to interpolate an extra-large pipe in a network of small ones.

14. Numerical Illustration.—In Fig. 3 is shown a typical town, lying on both sides of a stream and divided into eight sections by dotted lines. The elevations, referred

Fig. 3

to the adopted datum, are shown by figures in parentheses. The lengths of the lines are also shown. It is required to find the proper size of pipes to serve such a town, assuming the population to be 50,000, and the water consumption to be 19 cubic feet per second.

The quantities to be supplied to the different sections depend on the relative density of the population and the location of the manufacturing interests, and will be assumed to be $\frac{1}{19}$ of the total supply for section *1*; $\frac{3}{38}$ for section *2*; $\frac{2}{19}$ for section *3*; $\frac{5}{38}$ for section *4*; $\frac{3}{19}$ for section *5*; $\frac{5}{38}$ for section *6*; $\frac{4}{19}$ for section *7*; and $\frac{3}{19}$ for section *8*. The large main *A*, 10,000 feet long, conveys the entire volume of 19 cubic feet per second to the first point of branching, from which the submains *B* and *C* branch off. *B* extends 6,000 feet, and must deliver 4 cubic feet per second at an elevation of 550; *C* extends 4,000 feet, and must deliver 3 cubic feet per second at an elevation of 350. Although only a portion of these deliveries of 4 and 3 cubic feet reaches the extremities of the mains *B* and *C*, respectively, it is best in calculations of this sort to consider the whole volume as being delivered at the extremity and at the maximum elevation.

From the first point of branching, the main *J* extends 2,000 feet to the next two branches *D* and *E*, to which it must deliver 19 − 7 = 12 cubic feet per second. *D* delivers 3 cubic feet at an elevation of 400, through a length of 6,000 feet, supplying all that district lying between the river and the portion supplied by *B*; *E* has a length of 4,000 feet, and must deliver 2.5 cubic feet at an extreme elevation of 375 feet. The main *K* must supply 12 − 5.5 = 6.5 cubic feet through its length of 3,000 feet to the next point of branching. The branch *F* leading to the left has a length of 5,000 feet, a delivery of 2 cubic feet, and an extreme elevation of 375 feet; *G*, leading to the right, has a length of 5,000 feet, a delivery of 2.5 cubic feet, and an extreme elevation of 350 feet. The main *L* carries the remaining 2 cubic feet through 2,000 feet of pipe to the last two branches *H* and *I*. Each of these has a length of 4,000 feet and an elevation of 400 feet at the extremity; the delivery of *H* is 1 cubic foot, and that of *I* is 1.5 cubic feet per second. It is now necessary to determine the sizes of the mains. This problem is most readily solved by the use of the table for cast-iron pipes given in *Hydraulics*, Part 2.

The branch B has an elevation of 550 feet at the point O_1 of branching; therefore, the piezometric elevation must be greater than 550, so that water may flow from O_1 toward B. A 24-inch pipe will first be tried for the main A. Referring to the table, it will be found that, for a diameter of 24 inches and a discharge of 19 cubic feet per second, the value of s, or $\frac{h}{l}$, is .0052, and, since $l = 10{,}000$, this gives $h = 10{,}000 \times .0052 = 52$ feet as the required head between R and O_1. As this is greater than the actual difference in elevation $(600 - 556)$ between R and O_1, the assumed diameter is too small. Trying a 30-inch pipe, the value of $\frac{h}{l}$ is found to be .00167; therefore, $h = 10{,}000 \times .00167 = 16.7$ feet. This makes the piezometric elevation at O_1 $600 - 16.7 = 583.3$ feet. As this is greater than 556, the elevation of O_1, and also greater than 550, the elevation of B, the 30-inch pipe may be used for the main A. The heads for pipes B and C are, respectively, $583.3 - 550 = 33.3$, and $583.3 - 350 = 233.3$. The corresponding values of $\frac{h}{l}$ are $33.3 \div 6{,}000 = .00555$, and $233.3 \div 4{,}000 = .0583$. Knowing these values, and the discharges, the diameters can be taken from the table. They are 14 inches for pipe B and 8 inches for pipe C.

In carrying the main to the next branch point O_2, the possibilities of choice of size are greater. But since the point H, 11,000 feet away, is at an elevation of 400, it is desirable to reduce the head as little as may be, and it will be assumed that an effective head of 50 feet will give necessary pressures without making the pipes too large. The effective head in J being 50 feet in 2,000, the value of $\frac{h}{l}$ is $50 \div 2{,}000 = .025$; and from the table, the pipe necessary to carry 12 cubic feet per second with this value of $\frac{h}{l}$ is found to be between 14 and 16 inches. Using the 14-inch pipe, the value of $\frac{h}{l}$ is .033; $h = 2{,}000 \times .033 = 66$ feet, and, therefore, the piezometric elevation at O_2 is $583.3 - 66 = 517.3$ feet.

Proceeding as for the branches B and C, the value of $\frac{l}{h}$ for E is found to be .0355, which, by the table, requires an 8-inch pipe; for D, $\frac{h}{l} = .0196$, which, by the table, requires a 10-inch pipe.

Still bearing in mind the elevation of 400 at H, an effective head of 50 feet will be assumed between O_3 and O_2, so that the piezometric elevation at the junction O_3 will be $517.3 - 50 = 467.3$. The pipe K, then, will have a value of $\frac{h}{l}$ of $50 \div 3{,}000 = .017$; and it is found by the table that, for a delivery of 6.5 cubic feet per second, a 14-inch pipe is a little too large; it may, however, be used. The table gives, for that pipe, $\frac{h}{l} = .012$, and, therefore, $h = 3{,}000 \times .012 = 36$ feet. The piezometric elevation at the junction O_2 is, then, $517.3 - 36 = 481.3$. Proceeding as before, it is found that each of the branches F and G requires an 8-inch pipe.

Assuming an effective head of 30 feet for L, the value of $\frac{h}{l}$ is $30 \div 2{,}000 = .015$, and the pipe L is found to be between an 8- and a 10-inch pipe. For the 10-inch pipe, and the delivery of 2 cubic feet per second, the value of $\frac{h}{l}$ is .0058; therefore, $h = .0058 \times 2{,}000 = 11.6$, and the piezometric elevation at O_4 is $481.3 - 11.6 = 469.7$. The branches I and H are found to require diameters of 8 and 6 inches, respectively.

15. Remarks.—The sizes just determined are consistent with each other, but the size of the pipe H is not entirely satisfactory for fire service. Although lines of 6-inch pipe 4,000 feet long are often laid, it would be better to have the size increased to 8 inches. This would permit the head at the junction O_4 to be reduced, whereby all the heads and sizes of the system would be changed. Furthermore, it would be possible to have a pipe cast purposely 26 or 27 inches in diameter to act as a supply main from the

reservoir. Both changes should be worked out, and the costs of the different schemes compared, basing the costs on the weight of the different sizes of pipe.

The actual drafts on any part or all parts of the system laid out as has been described cannot be determined with any considerable degree of accuracy, because it is varying from moment to moment, and does not obey any fixed law. By following out the method just illustrated, however, a distribution and pressure of water that will always be effective and satisfactory can be secured. These calculations should invariably be made in any important project for piping a city, bearing in mind and providing for probable growth.

In cases where the topography of the town shows very great differences of elevation in certain sections, it may be necessary to divide the town into two or more zones, each furnished with a small distributing-reservoir tank or standpipe; or to have the pressure in the lower sections controlled by an automatic regulator that reduces the pressure as the water passes into the lower zone.

CONSTRUCTION OF PIPE LINES

PIPE LAYING

16. Distribution and Handling of Pipe Lengths. In a new water-supply system, the work is usually done by contract; in extensions to an existing system, the construction is usually done by day's work, under the direction of the waterworks superintendent. In either case, the pipe should be carefully unloaded from the cars and delivered on the opposite side of the street from where the excavation is to be made. The bells should all point in the direction in which the construction is to be carried on, and the several pipe lengths should be placed end to end, so that each length will be approximately opposite the place where it is to be used. Great annoyance and expense can be avoided by the

observance of these practical rules. When pipe has to be rolled by five or six men diagonally up or down the street, and then turned end for end before it can be lowered into the trench, and this operation has to be continued throughout the work, the importance of the instructions just given is evident.

Small sizes are handled by long poles inserted into the ends of the pipes, and may be unloaded easily from the wagon by dragging the pipe lengthwise until the rear end falls to the ground; then, starting the team, the forward end is allowed to fall on a bag of hay, to prevent breaking. In this way, the pipes are very readily placed end to end.

17. Inspection.—Pipes are inspected either by the *direct-observation test* or by the *hammer test*. The **direct-observation test** includes a close examination for fracture or cracks, for lack of coating, for blisters, sand holes, and other imperfections that may be detected by the eye. The **hammer test** consists in striking the pipe a sharp blow with a short-handled round-faced hammer weighing 3 or 4 pounds. If a pipe is cracked, a very characteristic harsh, jarring sound is heard; if not, a true ringing sound will follow the blow of the hammer. An experienced inspector can tell by the sound of the hammer blow something of the area and depth of the sandholes, and also the character of the coating. If the coating becomes loosened under the hammer blows, and exposes the metal beneath, the coating is not well laid, and the pipe, if used, should receive a field coating of hot asphalt.

The hammer test is not used on riveted steel pipe, for which dependence is placed entirely on observation and on the final water-pressure test. Such pipes may be deficient in proper riveting. A hammer blow on a rivet indicates whether the rivet is tight or not, and loose rivets may be cut out by chisels and redriven.

When cast-iron pipe of a certain weight is called for, the loaded wagons should be frequently weighed on a set of tested scales, to check the foundry weights, which are usually marked with white paint on the pipe.

18. Excavation.—Before beginning excavation, the line of the trench should be staked out with center stakes driven 100 feet apart on tangents, and 25 to 50 feet on curves. In paved streets, or where the surface is hard, iron spikes may be used. In cities where curbs have been laid, it may be sufficient to make marks on the curb, and indicate the distance from the curb to the center line. Stakes should also be driven at points where **specials** (that is, valves, crosses, tees, hydrants, etc.) are to go, so that their position may be indicated and their insertion in the line may not be forgotten.

Where the work is done by contract, it is most satisfactory to prepare a map, not necessarily to scale, showing the curb lines and the distance therefrom to the center line of the trench, and also the positions of all the specials. This map

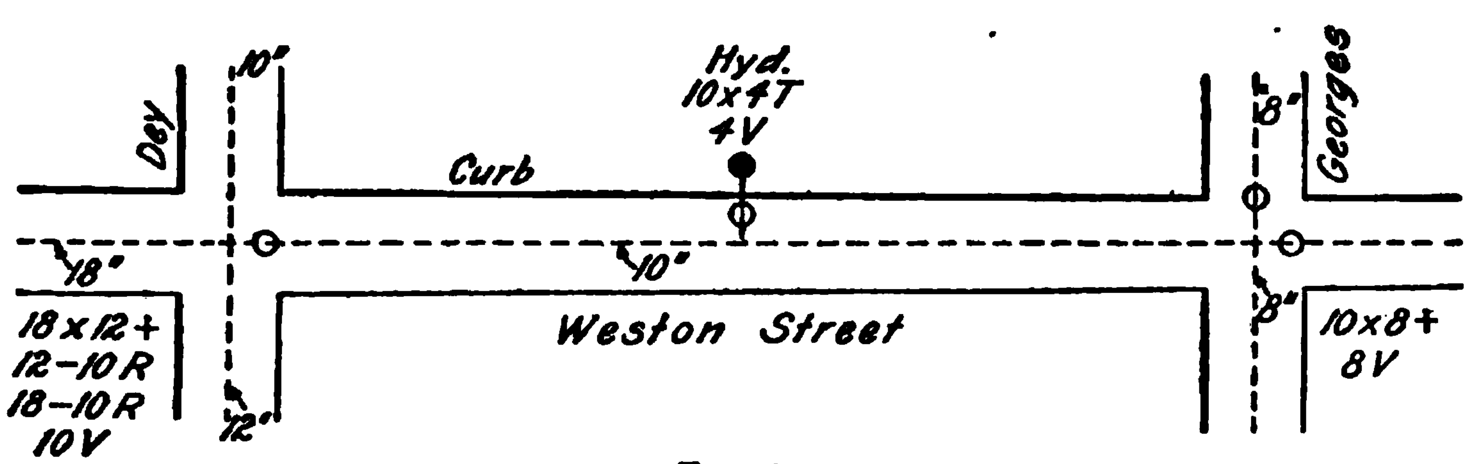


FIG. 4

is useful in distributing the pipe and specials, since it shows at once where each size will be needed. It is useful to the foremen who are laying the pipe, as it shows them what sizes and pieces are to be laid. Besides, it relieves the inspector or engineer of the need of being on the ground all the time, and makes the contractor responsible for placing the pipe and specials where they belong. A map of this kind is represented in Fig. 4. A cross indicates a crossing of two pipes at right angles; *T*, where one pipe enters another; *V*, a valve; and *R*, a reducer for changing gradually from pipe of one size to pipe of another size.

19. Excavation is usually paid for by the running foot, since, for water pipes, the depth is constant and the only difference in cost comes from the difference in material to

be excavated. All material excavated is paid for at the same rate, except rock, for which it is common to pay an additional price per cubic yard. The price for excavation is generally understood to include both excavation and refilling, but the contract or agreement should be explicit as to just what is intended. Pipe is usually paid for separately by the running foot of pipe laid, though often one price is made to include excavation, pipe, laying, refilling, and resurfacing. Appurtenances, such as valves, hydrants, blow-offs, etc., are paid for at an additional price, but the tees, crosses, and reducers are usually included in the length of pipe laid.

In excavation, the surface material, whether it is pavement, gravel, or unimproved surface, should be thrown out of the trench on the side of the street on which the pipe has been unloaded, and the rest of the excavated material thrown away from the pipe, so that the latter will not have to be lifted over the pile. The price of excavation may or may not include the cost of replacing the pavement, and this matter should be thoroughly understood before work is begun. Care must be taken that stones, dirt, and sticks do not enter the pipe during the progress of the excavation, and, except where pipe is being laid, the open end of the pipe should be plugged with a wooden plug set lightly in place. Refilling must be delayed until the pipe has been tested, although some earth may be thrown on the middle of the pipe to hold it in place.

20. Depth of Pipes.—Water pipe should be laid deep enough to be protected from the action of frost, otherwise serious cracks and breaks may occur. This is particularly likely to happen in small pipes with low velocity. Instances could be cited, however, of pipes crossing bridges with no protection whatever, freezing being prevented by the large size of the pipe through which water continually flows. In the northern American states, the usual specifications call for a 6-foot covering over the pipe, while in the central states, a 5-foot cover is considered sufficient. The texture of the ground has much to do with this, however, an open

porous gravel keeping the frost out much better than stiff clay. In the southern American states, at least 2 feet of cover should be provided, in order to protect the pipe against the great variations of temperature as well as against injury from heavy traffic.

21. Laying Cast-Iron Pipe.—The process of pipe laying may be described as follows: Two men are each given about 25 feet of 1-inch manila rope; they lay out 8 or 10 feet of this rope at right angles to and a short distance from the trench. On this the pipe is rolled close to the edge of the trench. The two men then plant their feet on the rope, back of the pipe (on the farther side from the trench), pass the other part of the rope around the pipe, and, by paying out the rope, slowly lower the pipe into the ditch; then, picking up both ends of the rope, and straddling the bank, they lift the pipe to the exact point where the spigot end enters the bell end of the last pipe laid. For pipes too large for men to hold or lift as described, small derricks are used. Just before inserting the suspended pipe, a fourth man in the trench wraps a few strands of oakum around the pipe, a special kind of hemp being prepared for this purpose. After the pipe has been entered the full length of the joint, the man in the trench, called the **yarner**, proceeds to force or calk more oakum into the joint, at the same time centering the pipes, so that there is the same space on all sides of the circumference, using steel wedges on the under side if necessary. The oakum is packed in until only enough depth of the joint is left for the lead; this depth varies for the different sizes, ranging from $\frac{3}{4}$ to $2\frac{1}{2}$ inches.

22. Joints.—Cast-iron pipe is usually made in pieces, called **sections** or **lengths**, about 12 feet long. Fig. 5 shows one-half of a longitudinal section PQ of one of these lengths. The section is broken at C, so as to show the principal parts to a large scale. The narrow end S of the pipe is called the **spigot**; the enlarged end AB, the **bell** or **hub**. The spigot S' of one length fits loosely into the bell of the other, the space RT being filled partly with oakum

and partly with lead. As will be observed, the length of each section is measured from the end of the spigot to the bottom of the bell, which serves as a seat for the end of the next section. The depth *b* of bell varies between 3 and

Fig. 5

6 inches, according to the diameter of the pipe; and the width *w* between the bell and the spigot varies between .4 and .5 inch.

23. As already stated, the joints in cast-iron pipe are made, or finished up, with lead. The lead is bought in "pigs" weighing about 100 pounds each, and is melted in a lead furnace. Coke is the most desirable fuel, though charcoal is often used, and coal or wood will answer. As soon as the lead is melted, all the "dross," or impurities, that rise to the surface, should be removed with a long-handled ladle. The molten liquid should be sufficiently hot to run freely after being transferred from the furnace to the ladle, and thence to the joint. No rule can be stated for the temperature; the iridescent appearance of the surface of the lead in the kettle soon teaches the melter the proper temperature.

Fig. 6

The calker prepares for the pouring of the lead by placing close to the hub a patent jointer, such as is shown in Fig. 6, having an opening through which the lead is poured.

Formerly, clay rolls were used, made of a rope of oakum, well daubed and smoothed with fireclay. Such rolls, however, do not hold the weight of the lead at the bottom, and the joint loses the lead when the latter is about half poured. The lead hardens in about 10 seconds: the jointer is then removed, and the calker cuts off any surplus lead that may remain at the point where the lead is poured in. Then, with specially prepared tools, the calker drives and forces the lead into the joint, compressing and packing it into all parts of the joint. In order to perform the work thoroughly, a wider opening in the trench is required, so that for this purpose it is necessary to dig bell holes around and under each bell.

24. Two things are indispensable for a good joint; namely, that the entire amount of lead necessary be poured at one time, and that the pipes be perfectly dry where the lead touches them. If water is present, it is instantly converted into steam by the hot lead, and the steam tears away the jointer and scatters the melted lead in every direction. If there seems to be no way of removing the water, as when a joint has to be made under the surface of the water, a cold joint may be made by taking a piece of lead pipe, cutting it to the right length, and calking it into the joint. This joint will answer for low pressures, but it cannot be made as firm as a poured joint, and is not to be used except where absolutely necessary.

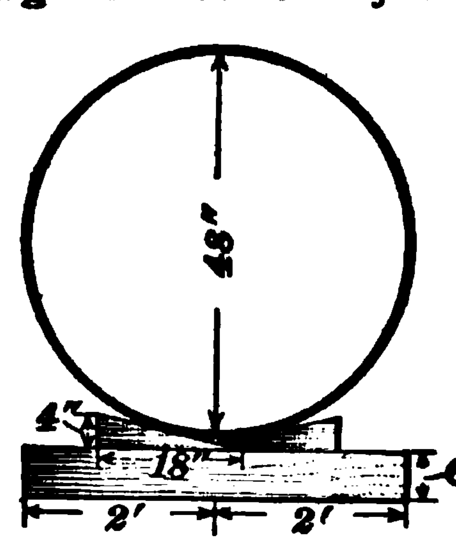


FIG. 7

25. Blocks and Wedges.—All pipe above 20 inches in diameter should be laid on blocks and wedged up. The blocks and wedges should be sawed out to regular dimensions. The blocks are laid a trifle below grade, and the pipe is raised to its grade by means of the wedges. Figs. 7 and 8 show a 48-inch pipe properly blocked and wedged. Fig. 9 shows the block and wedges to an enlarged scale. For smaller pipes, the blocks and wedges are lighter.

26. Weight of Lead per Joint.—The weight of lead necessary for each joint may be found by the following approximate formula:

$$L = .80\,d\,i$$

in which L = weight, in pounds, of lead in one joint;
d = diameter of pipe, in inches;
i = depth of joint, in inches.

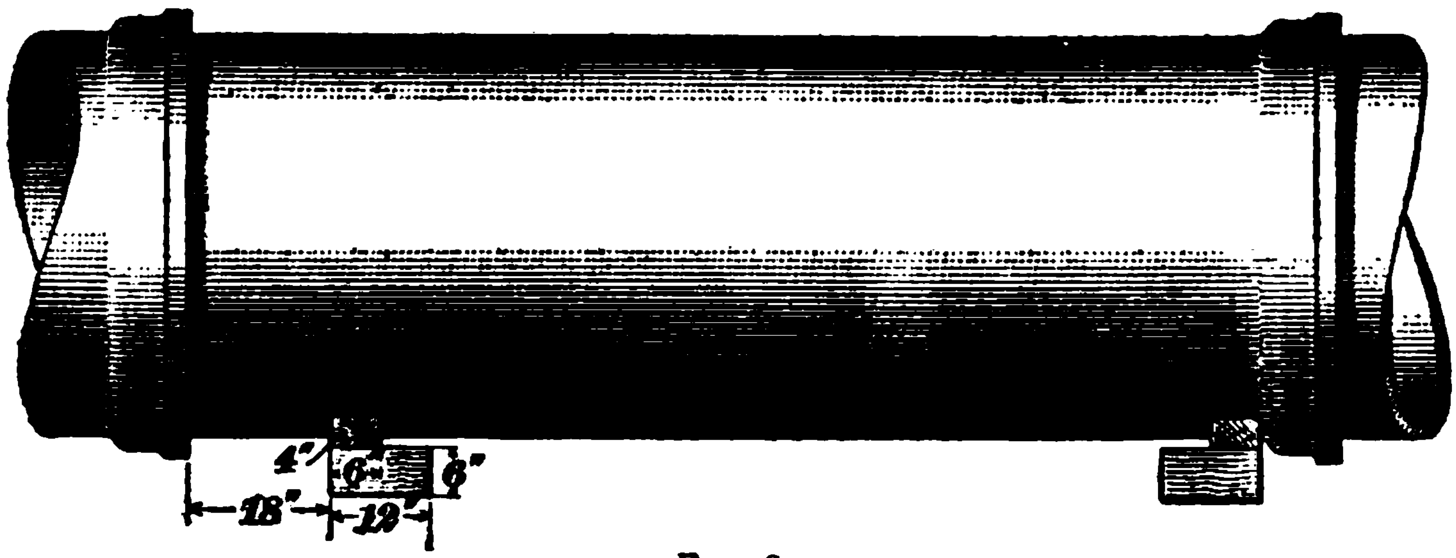


FIG. 8

EXAMPLE.—What weight of lead is necessary for a joint 4 inches deep, the pipe being 48 inches in diameter?

SOLUTION.—Substituting the given values in the formula,
$$L = .80 \times 48 \times 4 = 153.6 \text{ lb. Ans.}$$

27. Submerged Pipe Lines: Flexible Joints.—In cities where the source of supply is a river or a lake, or where a river crossing is necessary, either for a water or for a sewer main, the problem of laying a submerged main is often encountered. A dredge is usually employed to dig the

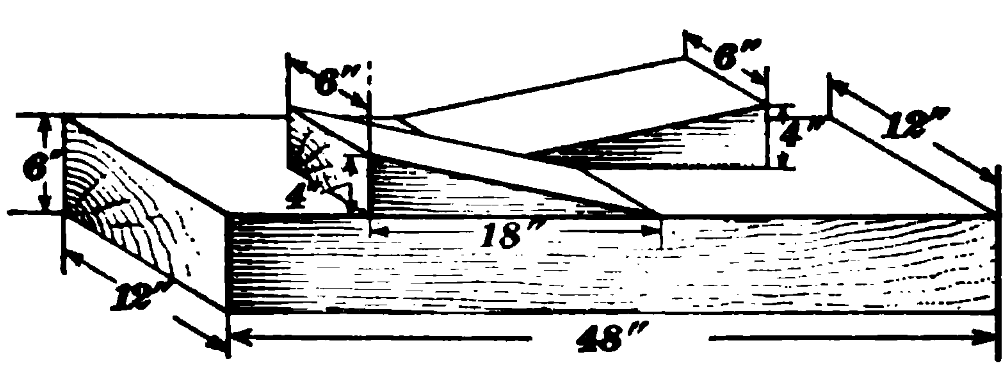


FIG. 9

trench, and then the pipe is lowered into it in one of several ways. The simplest method, where the depth is not great, is to build a series of trestles, simple **A** frames, across the stream over the prepared trench, and on these trestles joint

Front Elevation of Bent.

Fig

0

§ 85 20139

up the pipe line. Fig. 10 shows the design of the trestles and the relative position of the pipe, as used at Portland, Oregon. A bent is placed at every bell end, and the pipe is suspended by means of a ring attached to a long screw. When the line is completed, a man is stationed at each bent, and with regular turns of the screw the pipe is lowered into its position in the river bed.

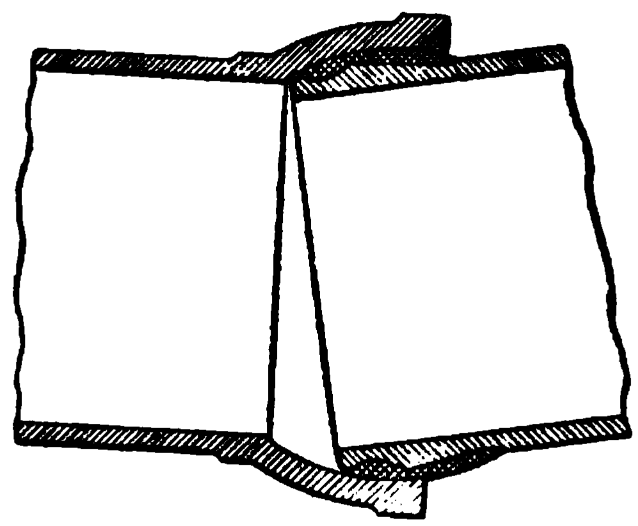


Fig. 11

28. In some cases, a submerged pipe line may be made up with the ordinary rigid lead joints, curves or bends being used for the bank approaches at either end. Irregularities of the river bed, however, or steep and irregular banks usually demand some form of **flexible joint.**

There are many designs of such joints, but perhaps the most popular and reliable is one devised by J. F. Ward, the principle of which is shown in Fig. 11. Another form, shown in Fig. 12, was devised by James Duane, and has been adopted as standard by the Croton Water Department of New York

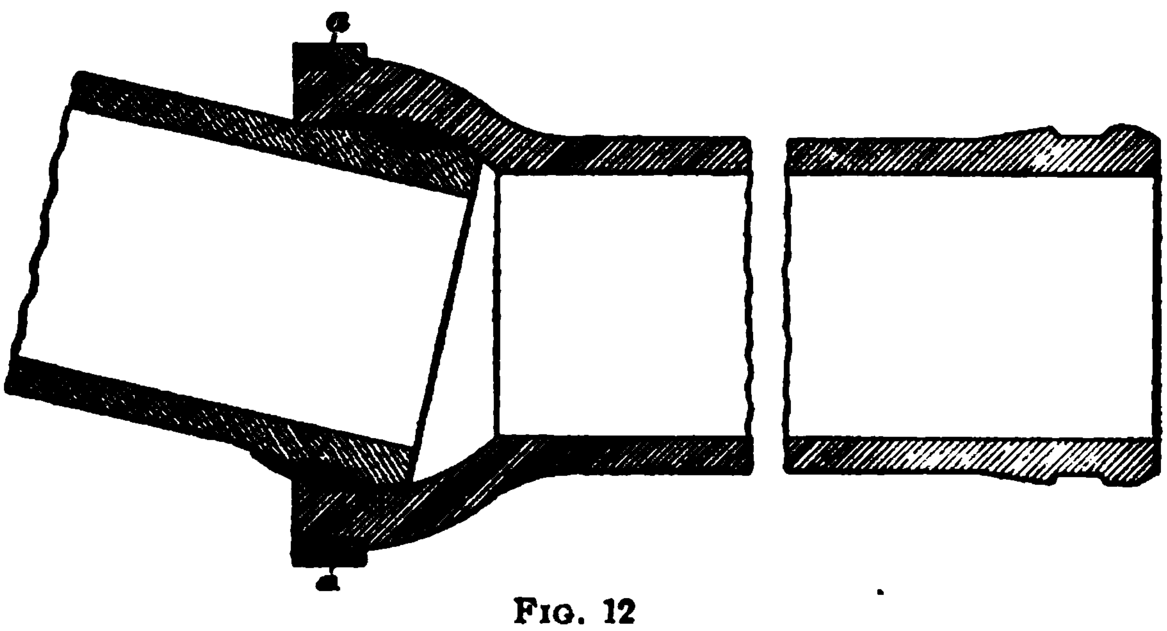


Fig. 12

City. It has a very rational design, and has been used for some years very successfully in difficult pieces of construction. Its special feature is the shrinking on of the wrought-iron band *a a*, which prevents the hub from cracking or splitting under any uncertain strain. As will be noted in the

figures, the motion of one pipe with reference to the next is limited, an angle of from 12° to 15° being the maximum provided for.

29. Advantage is often taken of the winter season to lay submerged pipe, the ice serving as an admirable platform on which to work. In this way, the cost of framing and falsework is eliminated, and hence the cost of the submerged pipe is much reduced.

30. In places where the pipe line is not too long, it may be laid by first driving a line of piles, about 30 feet apart, in a straight line across the river, at a given offset from the trench. The flexible joints are all made up on shore and connected in to the regular line of pipe. The flexible joints are used only occasionally, one flexible joint to every three, five, or ten regular lead joints, and the line is then floated out by being buoyed up by kerosene barrels, lashed in pairs on each side of the pipe at intervals. As the pipe is pulled across the river, it is kept in line by the piles. When the pipe reaches its final position, the barrels are cut loose, and it sinks to its place in the trench.

31. Another method is to provide a long scow with a guide frame at the end, sloping down into the water at such an angle that the flexible joints will follow without being broken. The pipes are jointed on the scow in the guide, and, the scow being drawn ahead as fast as a joint is made, the jointed pipe trails behind in place. Sometimes, two scows are fastened together to form a catamaran, and the pipes are jointed and trail into the water down a guide built between the two scows. Such a device is shown in Fig. 13.

32. In the simplest cases, where the water is not over 2 feet deep, a cofferdam of earth or timber may be made on each side of the trench, the trench dug, and the pipe laid in a dry trench, provided the soil is not too porous for the capacity of the pump that must be used to get rid of seepage water. This method may be followed, even where there is a current in the stream, by building one-half of the line at a time.

33. In all cases of laying submerged pipe, the pipe and joints should be carefully inspected before they are lowered into the trench. After they are in position, the open ends should be plugged, gauges attached, and air pressure applied. If there are any leaks, bubbles will appear on the surface of the water, indicating the location of the leaks. If the leaks are important, so that the air pump cannot hold the pressure required, a diver must be employed to examine the joints and recalk them.

34. Bridge Crossings.—Submerged pipes may be avoided when a bridge is available on which to support the pipe line. The disadvantage of this method of stream crossing is that the vibrations of the bridge, caused by passing loads, tend to loosen the joints in the pipe and cause frequent leaks, and that, unless the velocity is high, the water freezes if the pipe is not very carefully boxed and protected. Six or eight inches of porous non-conducting material, as dry leaves, tan bark, prepared insulating material, or even sawdust, is considered sufficient protection, though the exposure, size of pipe, and velocity of water are most important factors. If the pipe is large, it must be ascertained whether the bridge will carry the additional load of the pipe and its contents.

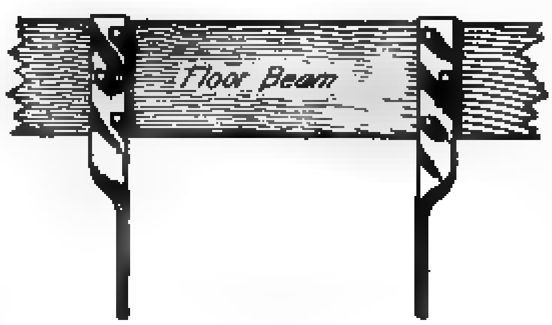


FIG. 14

35. Fig. 14 shows the usual method of construction for a bridge crossing. The pipe, box, and hangers are shown attached to the floorbeam of the bridge: the wooden block shown under the pipe is cut from a 2-inch plank sawed to proper dimensions, and placed back of the bell to support the pipe at each joint and lift it from the bottom of the box. The hangers are made from flat band iron, of a weight and thickness to support the box

properly, but in no case less than ⅜ in. × 2 in. The length of the hangers should be arranged to bring the box as close to the floorbeams as possible, in order to prevent swaying and swinging, and should be spaced to come at each joint or as near that spacing as the floorbeams will allow. The box is made of 2″ × 8″ pine, tongue-and-grooved, and thoroughly painted inside and out. This is covered with another course of ⅞-inch pine sheathing, also painted, and laid diagonally. The cover of the box should be made in sections to facilitate inspection and repairs.

36. The foregoing statements refer to cases most frequently encountered in building small water-supply systems, when a bridge is available. In small highway bridges, it is frequently possible to carry the pipe directly on the floor of the bridge, the pipe being protected as already described.

Because of the vibration, there is always leakage in a pipe carried by an ordinary bridge, and in many cases a special bridge for the water pipe can be built at less expense than a submerged pipe can be laid.

37. Curves in Pipe Lines.—It is frequently necessary to make in a pipe line a gradual horizontal or vertical curve, which is too long to be made with an ordinary bent pipe. The lead joint should be of equal width all around the circumference, but this is an ideal construction, and is not always practicable. There is always room enough in the

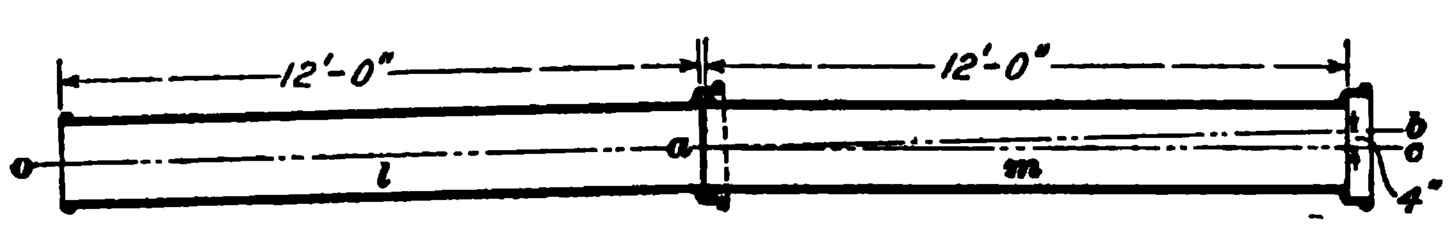


FIG. 15

joint to allow of a certain deflection from pipe to pipe, and in this way a slight curvature can be introduced. In Fig. 15 are represented two lengths of pipe, *l* and *m*. The axis of *m* may be moved so that it will diverge from that of *l* by 4 inches in a pipe's length, as shown. If this deflection is not enough to satisfy the requirements, shorter lengths may be used, cutting off from the spigot end as much as is

necessary. If 6-foot lengths are used, the deflection will be doubled; that is, the two axes will diverge by 4 inches in a distance of 6 feet, as shown in Fig. 16. A deflection greater than 4 inches does not leave enough room for proper calking of the joint on the outside, and, under high pressures,

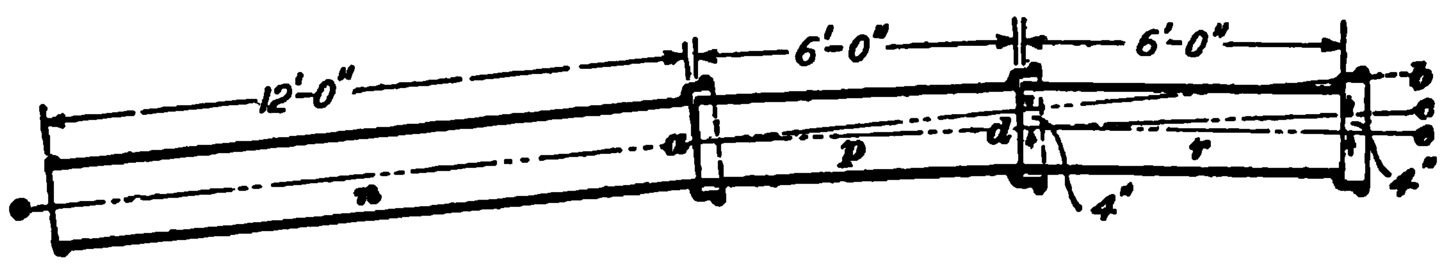


FIG. 16

leaks may be expected at that point. Sharp curves in a pipe should be avoided when possible; but when it is necessary to use them, special pipes, called **bends**, cast to a proper radius should be employed.

MISCELLANEOUS OPERATIONS

38. Removing Lead Joints.—It is often necessary to take up and relay a few lengths of pipe, and in some cases to remove several miles of pipe from a section of a city where the demand has outgrown the capacity of the pipe. To pull a leaded joint apart by steady pulling is almost impossible; to cut the joint out by hammer and chisel is very tedious and expensive. The usual method is to dig a hole around the joint and, using wood for fuel, burn out the joint. If water is present in the pipe or in the ditch, this method becomes impossible, and the pipes must be cut.

One of the simplest and most economical methods of removing lead joints is by the use of a device recently put on the market, in which a kerosene flame is thrown against the joint, the kerosene being delivered under pressure through about 25 feet of rubber tubing and a specially constructed nozzle or burner. When a little more than one-half of the pipe is uncovered, the flame with its intense heat may be concentrated on the upper portion of the lead joint, and in a few moments will raise the temperature of the lead to its melting point, when the pipes can be readily pulled apart.

By the use of this device, 8-inch pipes can be melted apart and put up on the bank at the rate of one every 7 minutes, and 16-inch pipes at the rate of one every 12 minutes.

39. Leaks in Pipe Lines.—A bad leak in a water main, whether the main is a force, a delivery, or a distributing main, is always a serious matter. It may cause much damage by overflow, or it may occur at such a time and place as to diminish materially the capacity of the main. In certain soils, as gravel, a leak is not easily located, and the escaping water may follow along the pipe for a long distance before making its appearance on the surface. Especially is this the case in a paved street. If the leak is at a joint, which sometimes blows out partly or entirely under heavy pressure, the joint must be recalked. If the leak is at a point where a house connection is made, a new connection is necessary.

If the body of the pipe is cracked or broken, the proper valves are shut and the water is drained or pumped out of that section. If the crack or break is short, a **split sleeve** may be applied (see Fig. 17). To apply the sleeve, a piece of canvas dipped in hot asphalt, or some other suitable gasket, is placed between the two flanges of the sleeve. Then the sleeve is bolted up tight, and the usual lead joint is made at each end of the sleeve. If the crack is a long one, the pipe must be cut out, preferably by the use of a pipe-cutting machine. A new pipe is then cut to replace the one taken out, and the ends are put together by sleeves, as just described. It is also possible, though it necessitates digging up 50 to 100 feet of pipe, to melt out three or four joints on each side of the break, and by raising the pipes in the form of an arch, spring in a new length to replace the one cut out.

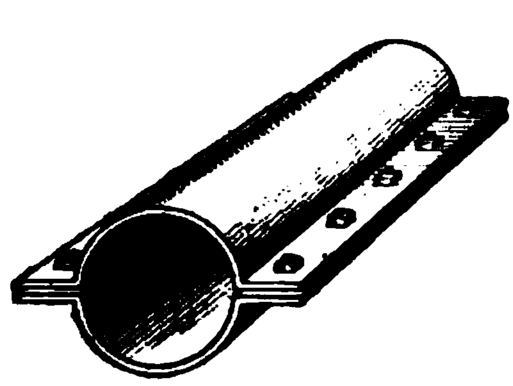


FIG. 17

40. Pipe Cutting.—In constructing a line of water pipe, it is always necessary to cut lengths of pipe in order to make proper connections with hydrants, valves, and

specials. This may be easily and quickly done by hand in

Fig. 18

the following way: A string is tied and a mark made with chalk around the circumference where the pipe is to be cut. The pipe is placed on a block of wood, as shown in Fig. 18.

Fig. 19

By following the chalk line, a groove is cut into the pipe by

means of a hammer and a cold chisel, as the pipe is revolved. This groove is uniformly deepened until, by repeated revolutions, the metal becomes weakened and the pieces fall apart, as shown in Fig. 19. If the pipe is defective and uneven in the casting, it may happen that it will not cut off square, but rather in an uneven manner, as shown in Fig. 20. Such a piece should not be used for the purpose intended, but may be cut again when a shorter length is needed; that is, it may be chipped down, as shown by the dotted line in Fig. 20. To do this, the piece of pipe is made to rest against an iron beam or convenient hard sharp edge, as shown in Fig. 21. By revolving the pipe and chipping off short pieces at a time, a comparatively clean-cut end may be secured.

FIG. 20

FIG. 21

41. Filling a System of Pipes.—When a system of piping, either a new system or one that for any cause has been emptied, is to be filled with water, the greatest care should be exercised in filling it. All the hydrants, blow-offs,

and air valves should be wide open, and the water admitted very slowly from the reservoir or in small quantities from the pump. It is hardly possible to go too slowly; and in an intricate system, several days may be required to fill it entirely. The reason for this is that the admission of large volumes of water into empty pipes causes a great agitation of the air that they contain, and induces oscillations in the water; the latter, by being suddenly stopped, produce violent shocks that may prove very injurious to the pipes. Hydrants, air valves, and blow-offs are closed as the water reaches them and begins to flow in a full, continuous stream, without sputtering. After the system is filled, the water should be allowed to stand for at least 24 hours, as a test for leakage.

SPECIAL CASTINGS

42. For the purposes of making connections, changing directions, passing from one pipe to another of different diameter, etc., special appliances, pipes, and combinations of pipes are used; they are known as **special castings**, or, simply, as **specials**. Of these specials, the most important are *reducers*, *tees*, *wyes*, *crosses*, *bends*, and *curves*. Specials are carried in stock by manufacturers, whose catalogs give the dimensions of the different parts.

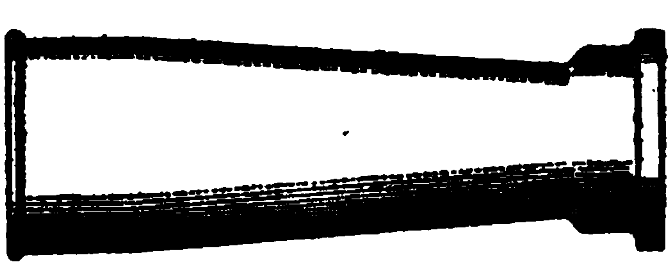


FIG. 22

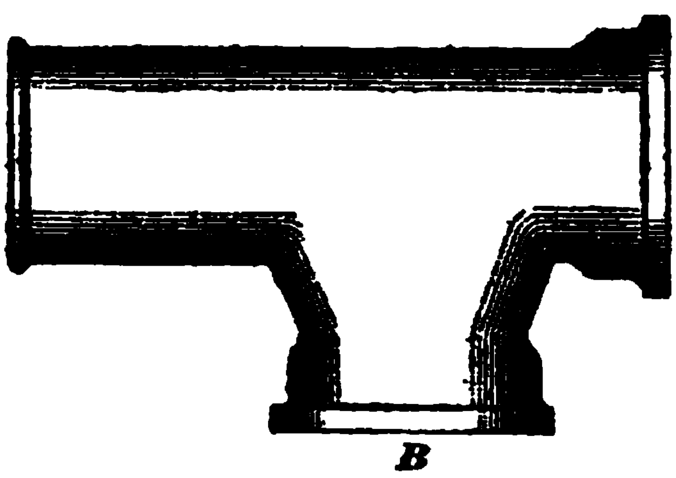


FIG. 23

43. A **reducer**, Fig. 22, is used for making a gradual connection between two pipes of different diameters. The bell end is fitted to the spigot of the smaller pipe, when the flow is from the larger pipe to the smaller. Otherwise, the bell end is placed in the larger end of the reducer, which then is often called an **increaser**.

44. A **tee,** Fig. 23, is used to make a right-angled con-

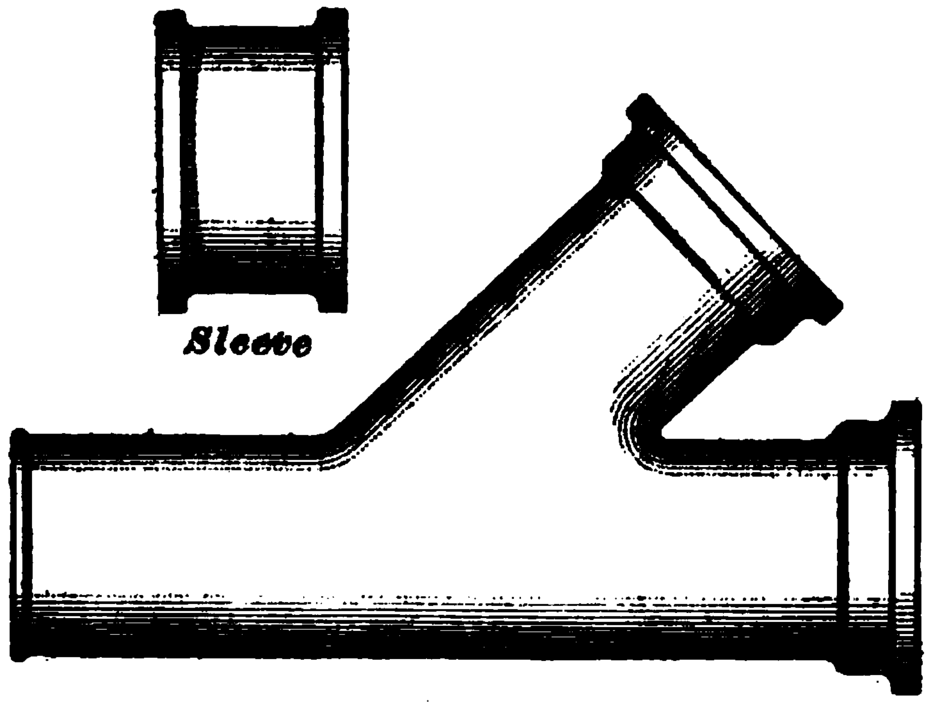


FIG. 24

nection between a main line and a branch, the spigot of the branch being jointed to the bell *B* of the tee. A **wye,** Fig. 24, is similar to a tee, except that it is used for oblique connections. A **cross,** Fig. 25, is a double tee, used where two lines branch off from the main line at the same point.

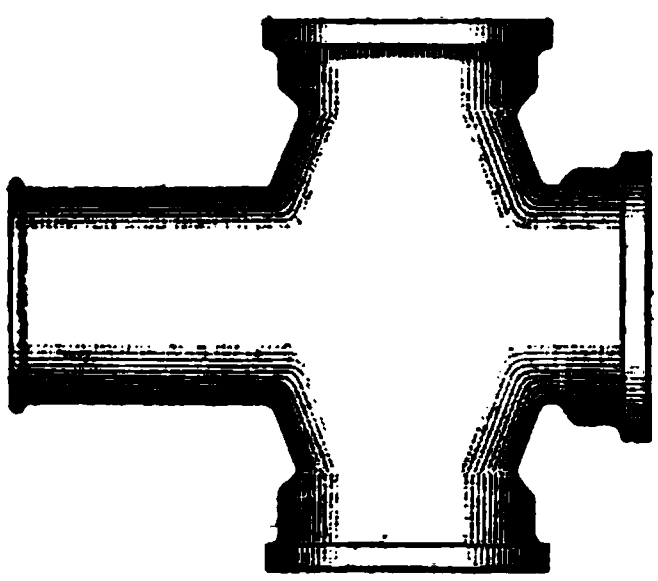


FIG. 25

45. A **bend,** or **elbow,** Fig. 26, is used for changing the direction of a pipe line at one point. Usually, bends are

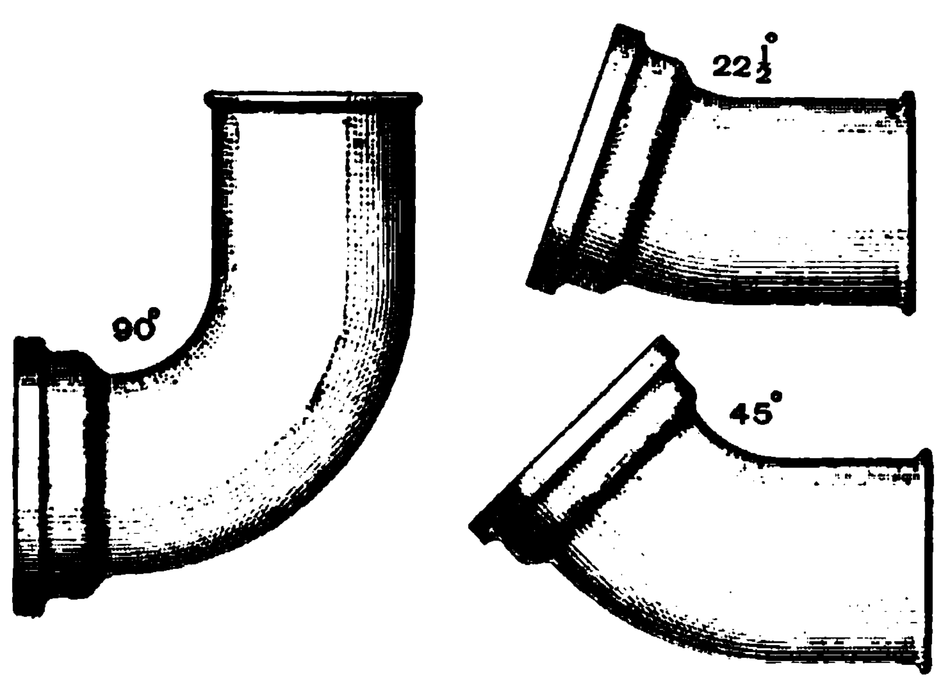


FIG. 26

indicated by the central angle of their circular part, as shown

in the figure. When the direction of a pipe line is to be continuously changed for some distance—that is, when the pipe line is on a curve—straight lengths may be employed, as explained in Art. **37**, or the change of direction may be effected by the use of **curves**, Fig. 27, which are curved lengths of pipe.

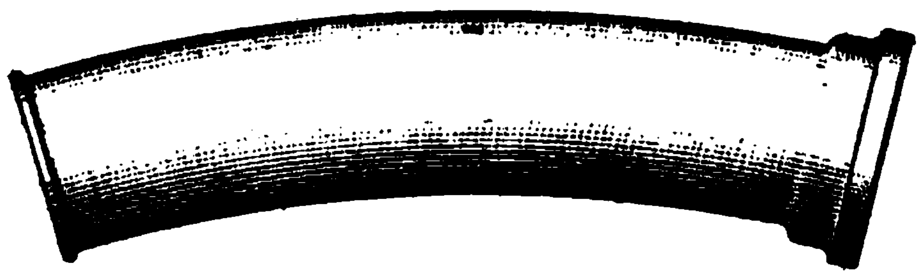


FIG. 27

HOUSE CONNECTIONS

46. House connections are made from the street main to houses by means of small pipes having an inside diameter of $\frac{1}{2}$ or $\frac{3}{4}$ inch. For this purpose, a tap is usually made into the top or side of the cast-iron main, and a connection of lead about 2 feet in length is made between the tap and the pipe running to the house, the inserted lead pipe being for the purpose of taking up, without injury, any possible settlement of either pipe. The tap is usually made when the pipe is in use, filled with water under pressure. This prohibits the use of the machinist's drill and tap, and recourse must be had to one of the many patented machines used for this purpose.

VALVES

47. Air Valves.—At all high places on the larger mains, especially if these places come near the hydraulic grade line, air valves should be placed, to permit the escape of air contained in the water, which tends to accumulate at high points. If some means is not provided for the escape of air, the pipe may become **air-locked**; that is, the volume and pressure of the air may reach such proportions as to diminish materially the area of the pipe available for water, and so reduce the discharging power of the pipe. In Fig. 28 is shown a sectional view of an air valve, the essential parts being the float F, the lever L, and the valve V. The valve is bolted or screwed to the pipe, with an opening between

the two at *O*. The weighted float *F* and the lever *L* are adjusted to a certain pressure, so that, when the chamber is filled with water (its normal condition), the poppet valve *V* closes tightly on its seat, and the lever and float take the positions shown by the dotted lines in the figure. When air begins to accumulate at the top or under the roof of the chamber, the water level is gradually lowered by the pressure of the air, and as the float descends toward the point *F*, it causes the valve *V* to open. In this way, the accumulated air escapes, but the float in rising closes the valve before any water

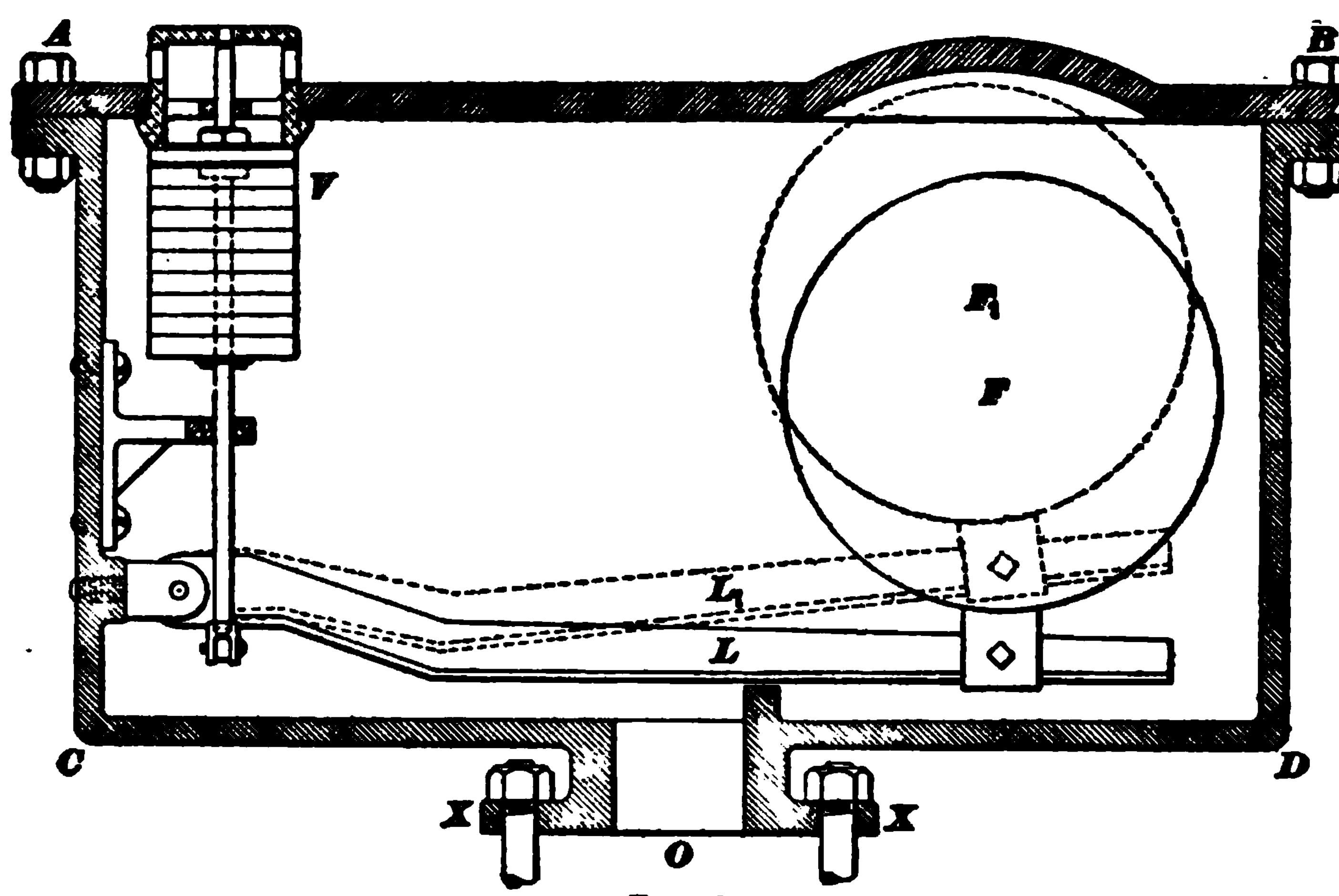


FIG. 28

follows, thus keeping the chamber always free of all but a small amount of air, and the pipe entirely free. The principle of this important device is similar to that of a boiler safety valve, except that the lever and weight are inside the chamber.

48. Blow-Offs.—At all depressions of the mains, and at all dead ends, **blow-offs** are necessary. They are special valves that are opened occasionally to remove silt and sediment by allowing the water to run out until it runs clear. A few lengths of sewer pipe may be laid to carry the water to some stream, gutter, or sewer where it will produce no nuisance. Hydrants are often used for the same purpose as blow-offs.

49. Shut-Off Valves.—In order that leaks or breaks in any pipe line may be repaired in case of accident, **shut-off valves** should be frequently introduced in the construction. On mains and submains, they should be placed at intervals of not more than 1,000 feet. Where mains or laterals intersect, valves should be placed on each side of the intersection. The governing principle is that the valves should be so placed that any length of pipe or lateral, of not more than two blocks in length, may be cut out of the general circulation by closing not more than four valves, and cause no interruption to the flow through the rest of the system.

(a) (b)

Fig. 29

This gives absolute control of every portion of the system; it makes possible the repairs to a break without shutting off the water from a long section of the line, which would temporarily inconvenience both domestic and commercial consumers, and place the whole district in jeopardy in case of fire.

Fig. 29 shows the general construction of an ordinary shut-off valve; the perspective view (*a*) shows the general form of construction, and (*b*) is a vertical section of the valve taken along the axis of the pipe. A wrench is applied

to the nut A, which on being turned to the right raises the valve v by means of the threaded valve stem B.

50. In large valves, the pressure against one side of the valve, when the water acts against that side and the pipe is empty or without pressure on the other, is often so great that the opening of the valve is very difficult. When heavy pressures are anticipated, a special valve is used having an auxiliary valve or by-pass through which a small stream of water can be allowed to pass around the main valve so as to equalize the pressures on the two sides. Large valves of this kind are usually set with the main valve stem horizontal instead of vertical, and are enclosed in a brick chamber built around the operating end.

51. Check-Valves.—The name **check-valve** is applied to an automatically acting valve that allows water to flow through in one direction and prevents it from flowing through in the other direction. These valves should be placed on pipe lines through which water is forced by a pump, so that, if the pump stops, the water will not run back to the pump. They are also placed at intervals of about 1,000 feet in long lines. In this way, not more than 1,000 feet of pipe can be drained, even if there is a break in the pipe.

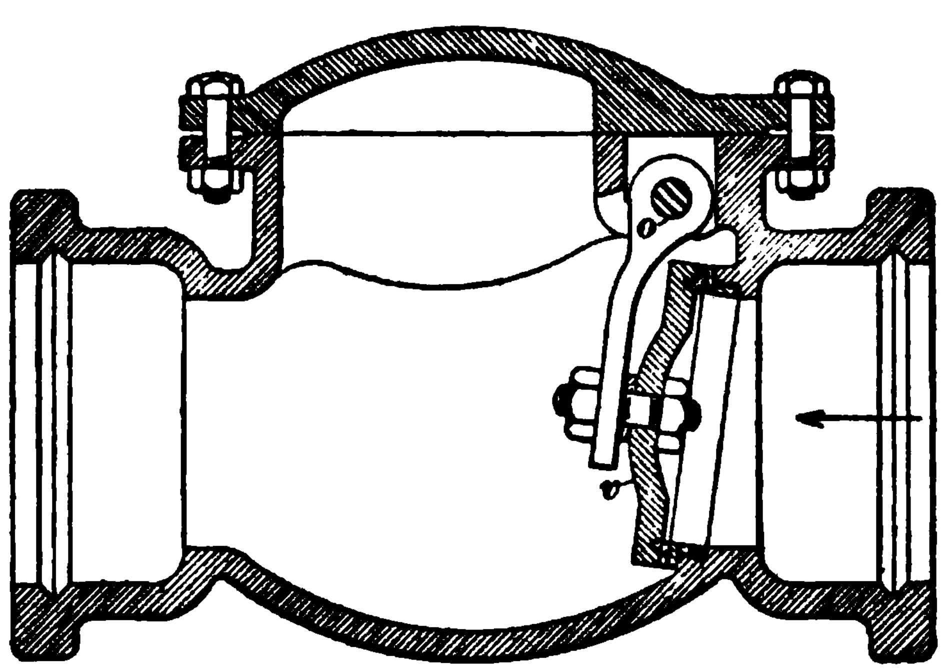


FIG. 30

A check-valve is shown in Fig. 30. Its operation is very simple: the valve v swings about the pivot o, and is raised by the pressure of the water flowing in the direction of the arrow; when flow in that direction ceases, the water tends to flow in the opposite direction, but is prevented by the valve, which drops to its seat and shuts off the passage in the pipe.

METERS

52. The Venturi Meter.—The **Venturi meter** is an appliance, remarkable alike for its accuracy and simplicity, used for measuring the volume of water flowing through a line of pipes. It is so named by its inventor, Clemens Herschel, in honor of the Italian physicist Venturi, who first noted, toward the end of the 18th century, the principle on which its action depends.

As shown in Fig. 31, the Venturi meter consists essentially of two conical tubes or reducers CC' of cast iron joined together at the small ends. The small ends are of

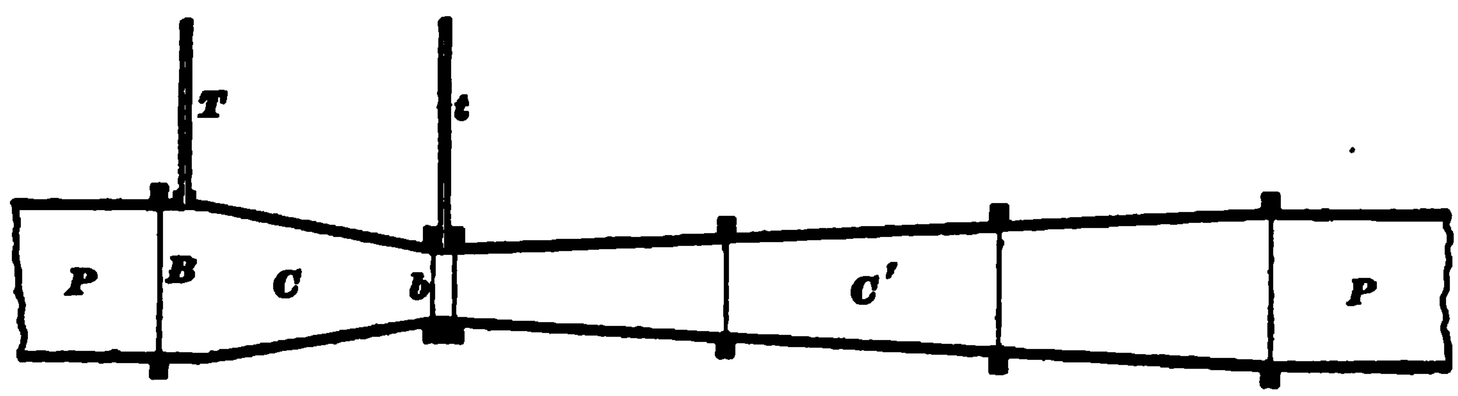


FIG. 31

the same diameter in both pieces, and the large ends are of the same diameter as the pipe PP, the flow through which it is desired to measure. The meter is placed at some convenient point in the pipe, so that all the water flowing through the pipe passes also through the meter.

Let piezometers T, t be inserted at the ends B and b of the tube C. Let the area of the end B, or of the pipe P, be denoted by A; the diameter of the pipe by d; the area at the end b by a; and the diameter at b by $\frac{d}{n}$. Also, let V and v, and H and h be, respectively, the velocities and piezometric heights at B and b. Since the flow is uniform, the sum of the pressure and velocity heads at B must be equal to the corresponding sum at b; that is,

$$H + \frac{V^2}{2g} = h + \frac{v^2}{2g};$$

or, because $\frac{v}{V} = \frac{A}{a} = \frac{d^2}{\left(\frac{d}{n}\right)^2} = n^2$, and, therefore, $v^2 = n^4 V^2$,

$$H + \frac{V^2}{2g} = h + \frac{n^4 V^2}{2g};$$

whence
$$V = \sqrt{\frac{2g(H-h)}{n^4 - 1}}$$

If the theoretical discharge, in cubic feet per second, is denoted by Q_0, then,

$$Q_0 = A V = \frac{\pi d^2}{4}\sqrt{\frac{2g(H-h)}{n^4 - 1}}$$

The actual discharge Q is obtained by multiplying Q_0 by a coefficient c determined by experiment:

$$Q = \frac{c\pi d^2}{4}\sqrt{\frac{2g(H-h)}{n^4 - 1}} = k\sqrt{H-h}$$

denoting by k the quantity $\frac{c\pi d^2}{4}\sqrt{\frac{2g}{n^4 - 1}}$, which is constant for any one instrument and can be computed once for all.

53. Although the Venturi meter can be used as just described, the discharge being computed by the formula given for Q, this is not the manner in which the instrument is generally used. It will be readily seen that it is possible to connect the tubes T and t with some apparatus that will record the difference $H - h$; instead, however, of graduating this apparatus to read values of $H - h$, it is so graduated that its readings give directly values of $k\sqrt{H-h}$, that is, values of Q. The graduations may also be made to read gallons per hour or per minute, etc. In some forms of the Venturi meter, a clockwork mechanism is used to keep a continuous record, so that the rate of discharge at any instant can be read from the instrument.

54. Service Water Meters.—There are many patented devices now on the market for measuring the flow of water in small quantities, especially in the service pipes of houses. The design or selection as well as the installation of these meters concerns the plumber rather than the engineer, and need not be treated at length here.

The accuracy of a water meter may be tested by weighing or measuring the amount of water passing through the meter

in a certain time and comparing the result with the reading of the meter. Errors of more than 2 or 3 per cent. in the amount registered seldom occur in meters sent out from the factory. Water meters should be *sensitive;* that is, they should be as accurate in measuring a small quantity as in measuring a large one. They should be *durable;* that is, they should not become inaccurate or non-sensitive after a year or two, but should continue to give good service for 8 or 10 years. Finally, they should offer as little *resistance* as possible to the passage of the water through them; this is of great importance where the water is pumped or where the pressure of the water in the mains is low. Meters are always tested before they leave the factory, and should be tested by the superintendent of the waterworks where they are to be used.

55. Water Waste and Prevention.—The subject of water waste has become very important in recent years, particularly in cities where the water has to be pumped. As shown in *Water Supply*, Part 1, the introduction of meters has the tendency to restrict waste. Defective plumbing and wilful waste, which are to be found in every city, often amounting to 40 per cent. of the entire delivery of the plant, may be checked by the general introduction of meters. Wherever meters are found, and the consumer pays for his water according to the amount he uses, just as he does for his gas and his coal, he is careful to avoid waste and defective plumbing. Another distinct advantage of the use of meters is that it places the superintendent in absolute control of the supply and enables him to distribute the cost per gallon in an equitable manner. It must not be inferred, however, that meters will correct all the waste in a water-supply system. Loss of water in mains and laterals is not uncommon where the pipe has been improperly laid, where corrosion and electrolysis have been at work, and where water hammer has opened joints and cracked the pipes. But the use of meters reduces if it does not entirely eliminate the careless waste from the domestic fixtures and the exorbitant uses of commercial establishments.

HYDRANTS AND FIRE-HOSE

56. Hydrants should be of the best grade and of approved design. They should have the waterway as unobstructed as possible, with easy bends, and with facilities for removing the valve and other interior mechanism without digging up the barrel of the hydrant. The operating valve may be a sliding gate or a compression valve; both types give good results. There should be places for at least two lines of hose to be attached, and three lines are not uncommon. The number of threads on the openings should exactly correspond to the threads on the hose couplings. The hydrant valve is opened and closed by the application of a wrench, which should fit exactly the pentagonal nut at the top of the valve stem, as well as the nut on the covers of the hose couplings. It should be specified that both valves and couplings should unscrew, or open, by turning to the left, and close by turning to the right. When the valve is closed, after the hydrant has been used, the barrel will be full of water, the valve being always at the bottom, and moved by a long valve stem reaching to the top. In order to prevent this water from freezing in cold weather, a drip or small opening is provided at the bottom, which is opened when the hydrant valve is closed, so that the water left in the barrel can drip away. In porous ground, this water simply leaches

FIG. 32

into the ground. In clay soil, it is best to connect the drip to the nearest sewer by means of a 1-inch pipe, or to a cess-pool dug around the hydrant and filled with broken stone or gravel. A 6-inch pipe is best to connect the hydrant with the main, though 4-inch pipe is often used. The connection pipe should have a valve by which the water can be shut off from the hydrant when the latter has to be repaired.

Fig. 32 shows the standard type of hydrant made by a prominent firm in the United States. *A* is the nut by which, through the screw at *B*, the valve at *C* is lowered or raised; *D* is the drip by means of which the water left in the barrel after the valve is shut is drained away.

57. The cost of the best hydrant is less than the cost of 100 feet of good hose, and hydrants should be placed near enough together to avoid the necessity of great lengths of hose. There should be a hydrant at every street intersection as a minimum number, and if the blocks are more than 400 feet long, one should be located halfway between corners. It is not advisable to locate a hydrant directly in front of a large building where heat or falling débris from a fire may cause it to become inoperative or inaccessible. It is better, when possible, to locate the hydrant about 50 feet away from large buildings.

58. The loss of pressure or of head due to water flowing through a hydrant follows the same general laws as were explained in *Hydraulics*, Part 1. The coefficient of resistance for each hydrant can, however, be determined only by experiment for each make of hydrant. C. L. Newcomb, in some experiments made in Holyoke, Massachusetts, found that, while the loss in pressure was four times as great in one make of hydrant as in another, the pressure lost in all cases was, when the hydrant was discharging at the rate of 500 gallons per minute, within the limits of .8 and 2.6 pounds, or 1.8 and 6 feet, which is a relatively small loss.

59. Fire-Hose and Nozzles.—The hose now generally used and considered the best for general fire service is 2½ inches inside diameter; it is made of heavy cotton fiber

TABLE I

PRESSURES REQUIRED FOR HYDRANTS

Diameter of Smooth Nozzle Inches	Pressure at Nozzle Pounds	Gallons Discharged per Minute	Vertical Height of Stream Feet	Horizontal Range of Stream Feet	Pressure at Hydrant or Pump Pounds									Diameter of Smooth Nozzle Inches
					50 Feet	100 Feet	200 Feet	300 Feet	400 Feet	500 Feet	600 Feet	800 Feet	1,000 Feet	
1	35	144	58	51	40	44	51	57	64	71	78	92	105	1
1	40	154	64	55	46	50	58	66	73	81	89	105	120	1
1	45	164	69	58	52	56	65	74	83	91	100	118	135	1
1	50	173	73	61	57	62	72	82	92	102	111	131	151	1
1	55	181	76	64	63	69	79	90	101	112	122	144	166	1
1	60	189	79	67	69	75	87	98	110	122	134	157	181	1
1	65	197	82	70	75	81	94	107	119	132	145	170	196	1
1	70	204	85	72	80	87	101	115	128	142	156	183	211	1
1	75	212	87	74	86	94	108	123	138	152	167	196	226	1
1	80	219	89	76	92	100	115	131	147	162	178	209	241	1
1	85	228	91	78	98	106	123	139	156	173	189	222		1
1	90	232	92	80	103	112	130	147	165	183	200	236		1
1	95	240	94	82	109	118	137	156	174	193	211	249		1
1	100	245	96	83	115	125	144	164	183	203	223			1
$1\frac{1}{8}$	35	185	59	54	43	49	60	71	82	94	105	127	149	$1\frac{1}{8}$
$1\frac{1}{8}$	40	198	65	59	50	56	69	81	94	107	120	145	171	$1\frac{1}{8}$

$1\frac{1}{8}$	45	210	70	63	56	63	77	92	106	120	135	163	192	$1\frac{1}{8}$
$1\frac{1}{8}$	50	222	75	66	62	70	86	102	118	134	150	181	213	$1\frac{1}{8}$
$1\frac{1}{8}$	55	232	80	69	68	77	95	112	130	147	165	200	235	$1\frac{1}{8}$
$1\frac{1}{8}$	60	242	83	72	74	84	103	122	141	160	180	218	256	$1\frac{1}{8}$
$1\frac{1}{8}$	65	252	86	75	81	91	112	132	153	174	195	236		$1\frac{1}{8}$
$1\frac{1}{8}$	70	261	88	77	87	98	120	143	165	187	209	254		$1\frac{1}{8}$
$1\frac{1}{8}$	75	270	90	79	93	105	129	153	177	201	224			$1\frac{1}{8}$
$1\frac{1}{8}$	80	280	92	81	99	112	138	163	188	214	239			$1\frac{1}{8}$
$1\frac{1}{8}$	85	288	94	83	106	119	146	173	200	227	254			$1\frac{1}{8}$
$1\frac{1}{8}$	90	296	96	85	112	126	155	183	212	241				$1\frac{1}{8}$
$1\frac{1}{8}$	95	305	98	87	118	133	163	194	224	254				$1\frac{1}{8}$
$1\frac{1}{8}$	100	313	99	89	124	140	172	204	236					$1\frac{1}{8}$
$1\frac{1}{4}$	35	230	60	59	48	57	74	91	109	126	142	178	212	$1\frac{1}{4}$
$1\frac{1}{4}$	40	246	67	63	55	65	84	104	124	144	164	203	243	$1\frac{1}{4}$
$1\frac{1}{4}$	45	261	72	67	62	73	95	117	140	162	184	229		$1\frac{1}{4}$
$1\frac{1}{4}$	50	275	77	70	68	81	106	130	155	180	204	254		$1\frac{1}{4}$
$1\frac{1}{4}$	55	289	81	73	75	89	116	143	170	198	225			$1\frac{1}{4}$
$1\frac{1}{4}$	60	302	85	76	82	97	127	156	186	216	245			$1\frac{1}{4}$
$1\frac{1}{4}$	65	314	88	79	89	105	137	169	201	234				$1\frac{1}{4}$
$1\frac{1}{4}$	70	326	91	81	96	113	148	182	217	252				$1\frac{1}{4}$
$1\frac{1}{4}$	75	337	93	83	103	121	158	195	232					$1\frac{1}{4}$
$1\frac{1}{4}$	80	349	95	85	110	129	169	208	248					$1\frac{1}{4}$
$1\frac{1}{4}$	85	360	97	88	116	137	179	221						$1\frac{1}{4}$
$1\frac{1}{4}$	90	370	99	90	123	145	190	234						$1\frac{1}{4}$
$1\frac{1}{4}$	95	380	100	92	130	154	201	247						$1\frac{1}{4}$
$1\frac{1}{4}$	100	390	101	93	137	162	211	261						$1\frac{1}{4}$

outside, and is lined with rubber inside. The cotton is woven in opposite directions in the two layers generally used to make up the body of the hose, to counteract the tendency to stretch under pressure. With careful use and thorough drying after service, the length of life of this hose is from 5 to 8 years. For ordinary use, a nozzle $1\frac{1}{8}$ inches in diameter has become practically standard.

60. The effectiveness of a fire stream depends wholly on the pressure and the quantity of water, so that the mains that supply the hydrants should be sufficiently large to deliver the requisite quantity of water with but little loss of head in the main, otherwise one stream working from one hydrant might so reduce the pressure that a second hydrant would deliver no water at all. The loss of head or pressure due to the water flowing through the hose is very great, and prevents the use of hose in lengths of over 400 feet, unless the hydrant pressure is great, and then there is danger of bursting the hose.

61. Table I, compiled from the valuable and elaborate experiments of John R. Freeman, gives the pressure required at the hydrant, in order to maintain certain pressures at the nozzles, for various lengths of $2\frac{1}{2}$-inch smooth, rubber-lined hose. The table shows that the pressure required to force water through more than about 400 feet with a pressure of 60 pounds at the nozzle is so great that, as already stated, it is preferable to avoid such great lengths by having hydrants at least as close together as 400 feet. A pressure of from 80 to 100 pounds at the hydrant when water is flowing from the hydrant is as much as a pipe line can readily withstand without continual leaks; with 200 feet of hose, this pressure will throw water to a height of about 75 feet, and will, therefore, be available for a fire in a six-story building.

EXAMPLE.—With a $1\frac{1}{8}$-inch nozzle, (*a*) what hydrant pressure is required to deliver 260 gallons per minute through 300 feet of $2\frac{1}{2}$-inch hose? (*b*) to what vertical height will the stream rise?

SOLUTION.—Consulting Table I, under the head of a diameter nozzle of $1\frac{1}{8}$ inches, we look for the given discharge of 260 gallons per

minute, and find 261 as the nearest number; following along the horizontal line until it intersects the vertical column whose heading is 300 feet, the length of hose, we find 143 pounds as the hydrant pressure.

(*b*) The vertical height to which a stream of water of 260 gallons per minute can be thrown with this hydrant pressure is found in the third column to be about 88 feet, which is the height that corresponds to a delivery of 261 gallons per minute.

EXAMPLES FOR PRACTICE

1. The Common Council of a city, in considering the introduction of a public water supply, demands that with eight lengths of hose, each 50 feet in length, and a $1\frac{1}{4}$-inch nozzle, a solid stream of water shall reach the roof of the city hall, which is 60 feet high. In the design of the system, what hydrant pressure must the engineer provide at this point to accomplish the required result? Ans. 109 lb.

2. In example 1: (*a*) what will be the discharge in gallons per minute? (*b*) what horizontal distance will the stream carry?

Ans. $\begin{cases} (a) & 230 \text{ gal.} \\ (b) & 59 \text{ ft.} \end{cases}$

KINDS AND PROPERTIES OF WATER-SUPPLY PIPE

CAST-IRON PIPE

62. Cast iron is the material most extensively used for water-supply pipes. The manner in which cast-iron pipe is made, and the methods of laying and joining the different sections, have been already described. The main advantage of this material is that its comparative freedom from corrosion makes it very durable. On account, however, of the limited length of which it is practicable to make the sections, and of the fact that, the tensile strength of cast iron being low and uncertain, inconveniently great thicknesses are required for heavy pressures, there is today a tendency to use steel instead, the latter material having, besides, the advantage of being cheaper. Steel pipe will be described further on.

63. Thickness of Cast-Iron Pipe.—Several formulas have been prepared for computing the thickness of cast-iron

pipe. The following formula, used by the Metropolitan Waterworks of Boston, seems to be one of the best:

$$t = \frac{(p + p')\,d}{6{,}600} + .25$$

in which t = thickness of pipe, in inches;

p = static pressure, due to the head above the pipe, in pounds per square inch;

d = diameter of pipe, in inches;

p' = allowance for **water hammer** (shocks caused by opening of valves).

The following are values of p' for different diameters:

Diameter of Pipe *Inches*	Value of p' *Pounds per Square Inch*
3 to 10	120
12	110
16	100
20	90
24	85
30	80
36	75
40 to 60	70

Example.—To determine the thickness of a cast-iron pipe 14 inches in diameter to withstand a pressure of 130 pounds per square inch.

Solution.—Here, $d = 14$ and $p = 130$. The value of p' corresponding to a diameter of 14 in. is a mean between the values corresponding to the diameters 12 and 16, or 105. Substituting these values in the formula,

$$t = \frac{(130 + 105) \times 14}{6{,}600} + .25 = .75 \text{ in. Ans.}$$

64. Weight of Cast-Iron Pipe.—Nearly all pipe foundries have catalogs giving the dimensions and weights of the pipes they manufacture. In Table II are shown the thicknesses and weights of standard cast-iron pipes for different diameters and pressures. Where no table is available, the approximate formula presently to be given may be used.

In calculating the weight of cast-iron pipe, each section is considered as being cylindrical from end to end, and 8 inches is added for the bell for all diameters. Thus, if a bell-and-

TABLE II

STANDARD THICKNESSES AND WEIGHTS OF CAST-IRON PIPE

Nominal Inside Diameter Inches	Class A 100-Foot Head 43 Pounds Pressure			Class B 200-Foot Head 86 Pounds Pressure			Class C 300-Foot Head 130 Pounds Pressure			Class D 400-Foot Head 173 Pounds Pressure			Nominal Inside Diameter Inches
	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	
4	.42	20.0	240	.45	21.7	260	.48	23.3	280	.52	25.0	300	4
6	.44	30.8	370	.48	33.3	400	.51	35.8	430	.55	38.3	460	6
8	.46	42.9	515	.51	47.5	570	.56	52.1	625	.60	55.8	670	8
10	.50	57.1	685	.57	63.8	765	.62	70.8	850	.68	76.7	920	10
12	.54	72.5	870	.62	82.1	985	.68	91.7	1,100	.75	100.0	1,200	12
14	.57	89.6	1,075	.66	102.5	1,230	.74	116.7	1,400	.82	129.2	1,550	14
16	.60	108.3	1,300	.70	125.0	1,500	.80	143.8	1,725	.89	158.3	1,900	16
18	.64	129.2	1,550	.75	150.0	1,800	.87	175.0	2,100	.96	191.7	2,300	18
20	.67	150.0	1,800	.80	175.0	2,100	.92	208.3	2,500	1.03	229.2	2,750	20
24	.76	204.2	2,450	.89	233.3	2,800	1.04	279.2	3,350	1.16	306.7	3,680	24
30	.88	291.7	3,500	1.03	333.3	4,000	1.20	400.0	4,800	1.37	450.0	5,400	30
36	.99	391.7	4,700	1.15	454.2	5,450	1.36	545.8	6,550	1.58	625.0	7,500	36
42	1.10	512.5	6,150	1.28	591.7	7,100	1.54	716.7	8,600	1.78	825.0	9,900	42
48	1.26	666.7	8,000	1.42	750.0	9,000	1.71	908.3	10,900	1.96	1,050.0	12,600	48
54	1.35	800.0	9,600	1.55	933.3	11,200	1.90	1,141.7	13,700	2.23	1,341.7	16,100	54
60	1.39	916.7	11,000	1.67	1,104.2	13,250	2.00	1,341.7	16,100	2.38	1,583.3	19,000	60

Note.—The above weights are for 12-foot lengths and standard sockets; proportionate allowance to be made for any variation therefrom.

spigot pipe measures 12 feet from the inside of the bell to the end of the spigot, it would be considered, in weight, as a cylinder 152 inches long.

Let W = weight, in pounds, of a length or section;
d = inside diameter, in inches;
t = thickness, in inches;
l = nominal length, in inches.

Then,
$$W = .82t(d+t)(l+8)$$

EXAMPLE.—A bell-and-spigot pipe, 16 inches in diameter and 12 feet in length, has a thickness of .70 inch. What is its weight?

SOLUTION.—Substituting the given values in the formula,
$W = .82 \times .70 \times (16 + .70) \times (144 + 8) = 1{,}457$ lb. Ans.

65. Weight of a Cast-Iron Pipe Line.—To ascertain by a rapid approximation the weight, in tons (2,000 pounds), of a cast-iron pipe line, the following formula may be used:
$$p = 28\,m\,t(d+t)$$
in which p = weight, in tons;
m = length, in miles.

In estimating, about 5 per cent. may be added to cover breakage, specials, and contingencies.

EXAMPLE.—What is the weight of 17 miles of pipe 16 inches in diameter and .7 inch thick?

SOLUTION.—Substituting given values in the formula,
$p = 28 \times 17 \times .7 \times (16 + .7) = 5{,}564$ T.
Adding 5 per cent., the required weight is
$5{,}564 + 5{,}564 \times .05 = 5{,}842$ T. Ans.

EXAMPLES FOR PRACTICE

1. A bell-and-spigot pipe, 14 inches in diameter and 12 feet long, has a thickness of .57 inch. What is its weight? Ans. 1,035 lb.

2. Determine, by Table II, the weight of a section of cast-iron pipe 16 inches in diameter, for a pressure of 86 pounds per square inch. Ans. 1,500 lb.

3. What is the weight of a pipe line 10 miles long, if the pipe is 18 inches in diameter and .75 inch thick? Ans. 3,938 T.

4. Determine the weight of 20 miles of pipe 16 inches in diameter and .7 inch thick. Ans. 6,546 T.

5. Determine the thickness of a 16-inch pipe to withstand a pressure of 86 pounds per square inch. Ans. .70 in.

66. Flanged Pipe.—For very high pressures, cast-iron flanged pipes are frequently used. A canvas gasket, dipped in hot tar or asphalt, is placed between the flanges, which are then drawn up tightly together by heavy bolts. Great care must be taken in the grade and alinement, as no deflection can be made at the joints. All the various specials used in the hub-and-spigot pipe are provided for flanged pipe, and every slight variation in line and grade must be made by

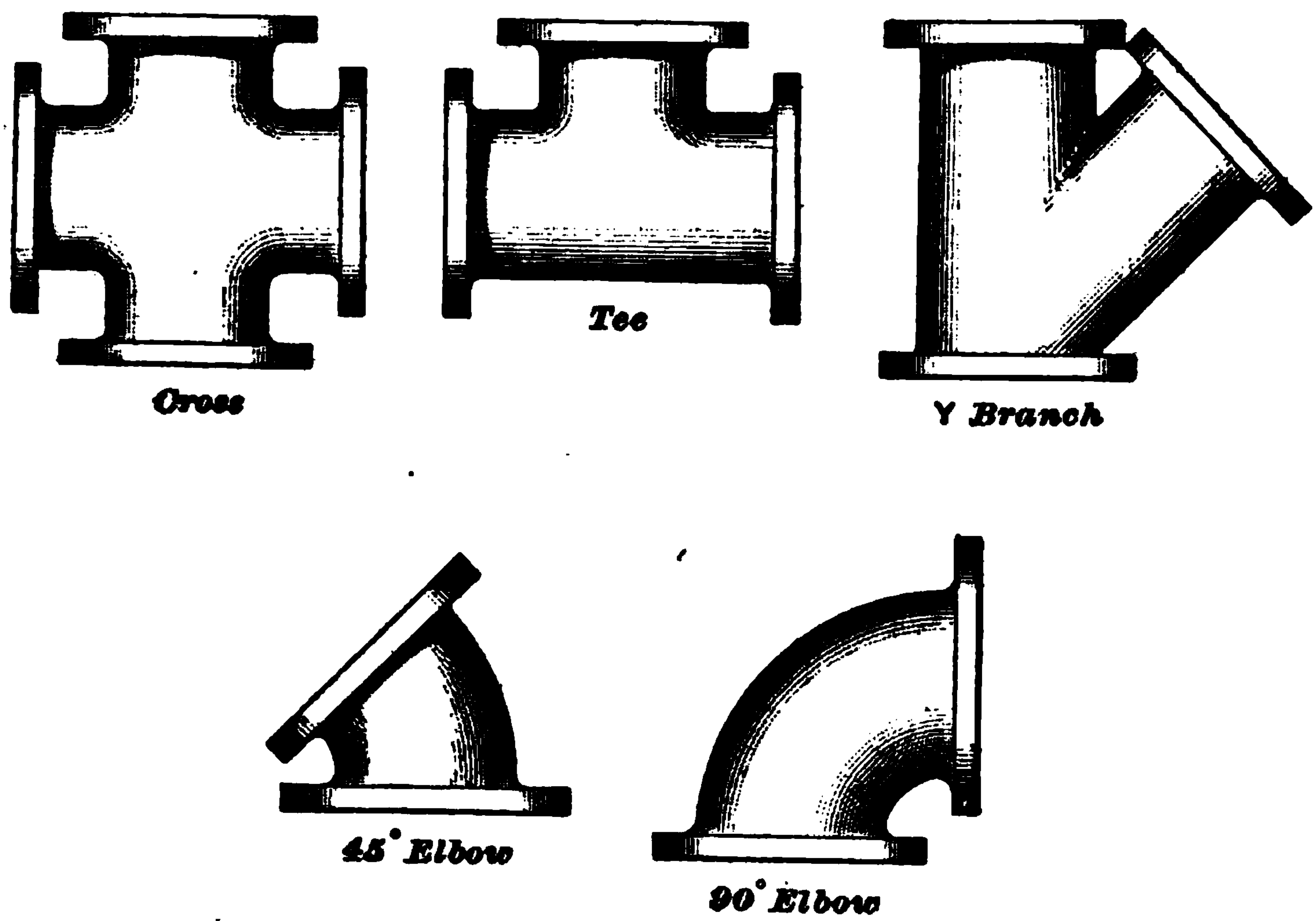


FIG. 33

their use. These pipes are designed to carry an actual pressure of as much as 400 to 500 pounds per square inch, and a large factor of safety is introduced to provide for water hammer and unequal settling in the ditch. They are thus heavier and more expensive than ordinary pipes, but are more satisfactory for high pressures. Fig. 33 shows various specials used with flanged pipe.

STEEL PIPE

67. General Description of Riveted Steel Pipe. At the present day, riveted steel pipes are extensively used, in many cases to great advantage. These pipes, which are

of sizes larger than 24 inches, are formed by riveting steel plates together in much the same manner as boiler shells are made. The rivet heads are either projecting or sunken; the sunken, or countersunk, heads improve the rate of flow, which the projecting heads impede. The sheets or plates are coated with some preserving material, usually asphalt, to protect them from corrosion. The coating is applied by immersing the sheets in asphalt heated to a temperature of about 250° or 300° F. The durability of steel pipes depends to a great extent on the coating; if good coating is used, and the pipe is thoroughly cleaned and recoated about every 10 years, steel pipe will be found as durable as cast-iron pipe.

The pipes are sometimes shipped in sheets and bent and riveted on the ground. This facilitates shipping and handling. Great difficulty, however, is found in securing good field riveting of the joints, and in having the protective coating as well applied in the field as in the shop.

Riveted pipes are made in lengths of from 24 to 36 feet, and therefore require fewer joints than cast-iron pipes. They have the advantage that they can be made of any diameter, in fractions of an inch if desired, without additional expense, and therefore can be made to conform exactly to the results of calculations.

68. Joints.—The lengths of riveted pipe can be connected in different manners, there being several special contrivances for this purpose. They are sometimes connected by taper joints, the external diameter of one end of each length being the same as the internal diameter of the other, so that one can enter the other and the two be riveted together. Cast-iron joint pieces are also used, to which the steel pipe is fastened.

69. Quality of Material for Steel Pipe.—The specifications for steel supply pipe should be the same as those for standpipes, and the same rules should hold for punching and riveting. The subject of standpipes is treated in *Water Supply*, Part 3. The longitudinal seams of the water pipe should be double-riveted; and the vertical seams, that is, those around the pipe, single-riveted.

70. Thickness of Riveted Steel Pipe.—The thickness of a riveted steel pipe may be computed by the following formula:

$$t = \frac{p\,d}{20{,}000} + .3$$

in which t = thickness, in inches;
d = diameter of pipe, in inches;
p = pressure, in pounds per square inch, due to static head.

EXAMPLE.—To determine the thickness of a riveted steel pipe 36 inches in diameter, to withstand a pressure of 125 pounds per square inch.

SOLUTION.—Here, p = 125 and d = 36. Substituting these values in the formula,

$$t = \frac{125 \times 36}{20{,}000} + .3 = .53 \text{ in.} \quad \text{Ans.}$$

71. Flow in Riveted Pipes.—On account of their special construction, riveted steel pipes offer greater resistance to flow than cast-iron pipes. Sufficient data are not available from which a satisfactory value for f (see *Hydraulics*) can be found. The formula most generally used for the velocity in riveted pipes is Kutter's formula (*Hydraulics*, Part 3), with a value for n varying between .013 and .015.

72. Laying Steel Pipe.—In laying steel pipe it is essential that the pipes should have a firm bearing in the trench, free from stones, boulders, or projecting pieces of rock. The riveting should be done as well as possible, the joints all carefully calked, and the rivets and other exposed surfaces painted with a protective paint. Great care should be taken to prevent workmen from injuring the coating originally applied to the pipe, and special precautions, such as requiring the riveters to wear rubbers, or to walk only on strips of carpet or canvas provided for that purpose, have been considered necessary. Great pains must be taken in filling in the trench; soil free from stones should be well rammed under the sides of the pipe and alongside it up to the top. Serious difficulties have arisen from failure to support the sides of light steel pipe by properly rammed earth

filling underneath, in consequence of which settling has occurred at the top or crown.

73. Lap-Welded Pipe.—Lap-welded pipe is made in stock sizes up to 2 feet in diameter, and in lengths of from 20 to 30 feet. The sections are provided with cast-iron joints, usually patented, by means of which joints are made, either by the ordinary lead joint used in cast-iron pipe, or by special devices that require hydraulic pressure or heavy calking to make them tight. Lap-welded pipe presents a very smooth interior surface, has fewer joints to the mile than cast-iron pipe, and would in all respects be preferable to the latter were it not for its rapid corrosion. It has an extended use, however, in some classes of work, where permanency of construction is not an essential feature.

74. Spiral Riveted Pipe.—Spiral riveted pipes are used to a large extent in irrigation work, and appear to be economical and to give good service for a limited time. They are made of wrought iron or steel in various thicknesses, riveted spirally, as shown in Fig. 34, and provided

FIG. 34

with a cast-iron head by which the joints are made as with a flanged pipe. The pipes can be made of any diameter and to withstand any pressure. The projecting rivet heads reduce the capacity of the pipe to about 85 per cent. of the capacity of a cast-iron pipe of the same diameter.

WOODEN-STAVE PIPE

75. General Description.—Pipes composed of wooden staves, held together by round steel rods, called **bands**, have been used for a long time in irrigation works, and have found favor there on account of their cheapness. Owing to improved methods of manufacture, their use is being extended to water supplies under considerable pressures. In the smaller diameters, the pipes are often made up at the works, where they are machine-banded and provided with special connecting castings, and are shipped in 30-foot lengths to the place where they are to be used. In the

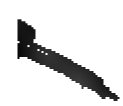


FIG. 35

larger diameters, up to 10 feet, the pipes are built on the ground, the staves and bands being supplied. The staves are laid so as to break joints, and the ends are fitted with steel tongues that are embedded in the butting joints of each two continuous staves.

Curves and long bends can be made by bending the staves, and all ordinary connections, such as branches, tees, and valves, are made by fastening such appurtenances to the pipe by bands or bolts. Fig. 35 shows the usual construction.

Each band is tightened and held in place by a coupling shoe, of which an enlarged view is shown at S. The best results so far obtained have been from California redwood and Oregon fir, well seasoned and cut into lengths of from 8 to 20 feet.

While it is improbable that wooden pipes, such as have been described, will ever replace cast-iron or steel mains for town or city use, yet they will doubtless be employed for carrying water for long distances and in quantities that necessitate large diameters. Their cheapness in first cost, in transportation, and in laying will lead to their use in cases where iron and steel are precluded on account of their cost. Other advantages of wooden pipes are that they are free from tuberculation, and have a tendency to wear even smoother than when first made. On this account, the flow may be computed by using for the coefficient f the values applying to smooth iron pipe; it may be safely assumed that this value will hold, even when the pipes become old, provided, however, that the velocity of flow in the pipe is at least 2 feet per second, so that no fungus growths can form.

76. Formulas for Stave Pipe.—The following formulas may be used in the design of wooden-stave pipes:

$$d = \frac{D + 2t}{80} \qquad (1)$$

$$s = \frac{65\,(D + 2t)^2}{16\,(pD + 200\,t)} \qquad (2)$$

in which d = diameter of bands, in inches;
D = inside diameter of pipe, in inches;
t = thickness of pipe, in inches;
p = water pressure in pipe, in pounds per square inch;
s = distance between bands, in inches.

It is not advisable to use bands less than $\frac{3}{8}$ inch in diameter, as they are likely to cut into the wood. By reducing the distance between the bands, stave pipe can be made to stand very heavy pressures, but above about 85 pounds per square inch, the cost of construction is equal to or greater than for steel pipe.

Fig. 36

77. Dimensions of Staves.—Table III gives the dimensions of staves recommended by A. L. Adams, who has made a thorough study of stave pipes.

TABLE III

DIMENSIONS OF PIPE STAVES

Nominal Diameter of Pipe Inches	Stock Sizes for Staves Inches	Thickness of Finished Staves Inches
22	2 × 6	$1\frac{3}{8}$
24	2 × 6	$1\frac{3}{8}$
27	2 × 6	$1\frac{7}{16}$
30	2 × 6	$1\frac{1}{2}$
36	2 × 6	$1\frac{9}{16}$
42	2 × 6	$1\frac{5}{8}$
48	2 × 6	$1\frac{11}{16}$
54	$2\frac{1}{2}$ × 8	$2\frac{1}{8}$
60	3 × 8	$2\frac{1}{2}$
66	3 × 8	$2\frac{9}{16}$
72	3 × 8	$2\frac{5}{8}$

EXAMPLE 1.—To determine the diameter of the bands to be used on a wooden-stave pipe 48 inches in diameter, $1\frac{11}{16}$ inches in thickness.

SOLUTION.—Here, $D = 48$ and $t = 1\frac{11}{16}$. Substituting in formula **1**,

$$d = \frac{48 + 2 \times 1\frac{11}{16}}{80} = \frac{5}{8} \text{ in., nearly. Ans.}$$

EXAMPLE 2.—To determine the distance between bands in example 1, the pressure being 75 pounds per square inch.

SOLUTION.—Here, $p = 75$, $D = 48$, and $t = 1\frac{11}{16}$. Substituting in formula **2**,

$$s = \frac{65 \times (48 + 2 \times 1\frac{11}{16})^2}{16 \times (75 \times 48 + 200 \times 1\frac{11}{16})} = 2\frac{3}{4} \text{ in., nearly. Ans.}$$

78. Fig. 36 shows a wooden-stave pipe built by the Washington and Oregon Power Company, Walla Walla, Washington. The pipe is 48 inches in diameter, with reversed curves, on a radius of 200 feet. The great flexibility of wooden-stave pipe should be particularly noticed.

79. Laying Wooden-Stave Pipe.—In laying stave pipe, great care should be taken that the staves are in perfect condition. Those which are bent, twisted, cracked, or knotted should be rejected. The bands should be thoroughly coated and spaced according to the computed distances. Stakes should be driven along the line of the pipe to show where the spacing of the bands changes.

SERVICE PIPES

80. Service pipes, or simply **services,** are small pipes connecting the street mains with the house pipes. Although this subject belongs to plumbing rather than to engineering, it will be briefly considered here. Service pipes are very liable to corrosion and deterioration, from the character of the ground in which they are laid, and the best material from which to make them is not yet fully determined, though current practice is in favor of galvanized iron. Lead has been largely used, is more permanent than iron, and is not broken by uneven settlement of the pipe in the trench, nor is it so likely to be injured by frost in winter. Instances are on record of lead pipe, carrying pure water, being acted on by the water, and forming lead oxide, which is very poisonous. Hard waters do not form this poisonous compound, however, and probably there are but few waters so pure as to be dangerous after flowing through a lead pipe. Sometimes, lead pipe with a tin lining is used. Other materials and linings are occasionally employed; but, as a rule, galvanized-iron pipes $\frac{1}{2}$ or $\frac{3}{4}$ inch in diameter, according to the size of the house, are used for service pipes to residences.

GENERAL REMARKS ON PUMPING

81. Power Required in Pumping Through Mains. As explained in *Hydraulics*, Part 2, the resistance to flow in pipes is made up of three terms; namely, resistance due to friction, resistance at the entrance of the water into the pipe, and resistance due to bends, contractions, enlargements,

valves, and other obstructions in the pipe. In a long pipe, the resistance due to friction is the only one of importance, the others being entirely neglected in most cases. In pumping through mains, the resistances due to bends and other obstructions may, if the velocity is high, be sufficient to make it worth while to take them into account.

The units being the foot and the second, let h_1 represent the head lost in friction; h_2, the loss at entrance; h_3, the loss caused by bends, valves, etc.; and h, the height through which the water is to be pumped. Let U represent the work performed by the pump each second, and Q be the number of cubic feet of water delivered per second. Then, the weight raised per second is 62.5 Q; and the work is the same as if this weight were raised to a height equal to the actual height plus the heights represented by the resistances. Therefore,

$$U = 62.5\,Q\,(h + h_1 + h_2 + h_3)$$

Substituting the values of h_1, h_2, and h_3 given in *Hydraulics*, Part 2,

$$U = 62.5\,Q\left[h + \left(f \times \frac{l}{d} + m + k\right)\frac{v^2}{2g}\right]$$

or, because

$$v = \frac{Q}{A} = \frac{Q}{\frac{\pi d^2}{4}} = \frac{4\,Q}{\pi d^2}, \text{ and } \frac{v^2}{2g} = \frac{16\,Q^2}{2g\pi^2 d^4} = .0252\frac{Q^2}{d^4},$$

$$U = 62.5\,Q\left[h + .0252\left(f \times \frac{l}{d} + m + k\right)\frac{Q^2}{d^4}\right] \qquad (1)$$

Since this is the work, in foot-pounds, done per second, we have for the corresponding horsepower

$$\text{H. P.} = \frac{U}{550} = .1136\,Q\left[h + .0252\left(f \times \frac{l}{d} + m + k\right)\frac{Q^2}{d^4}\right] \qquad (2)$$

The value of f as an average is about .018; the value of m is found to vary from 0 to .93, and the value of k is about .9 for *each* bend, though it varies with the angle of the bend.

EXAMPLE 1.—It is desired to raise 15 cubic feet of water per second by pumping to a reservoir 300 feet above and 2 miles distant from the pumping well. What horsepower will be necessary to do this work through a main 24 inches in diameter, having four bends, assuming a value of .5 for m?

SOLUTION.—Here, $Q = 15$, $h = 300$, $l = 5{,}280 \times 2 = 10{,}560$, $d = \frac{24}{12} = 2$, $f = .018$, $m = .5$, and $k = .9 \times 4 = 3.6$. Substituting these values in formula **2**,

$$\text{H. P.} = .1136 \times 15 \times \left[300 + .0252 \times \left(.018 \times \frac{10{,}560}{2} + .5 + 3.6\right) \times \frac{15^2}{2^4}\right]$$
$$= 571 \text{ H. P. Ans.}$$

EXAMPLE 2.—Assuming the same data as in example 1, but the diameter of the pipe being 1.5 feet instead of 24 inches, to determine the horsepower.

SOLUTION.—Substituting the given values in formula **2**,

$$\text{H. P.} = .1136 \times 15 \times \left[300 + .0252 \times \left(.018 \times \frac{10{,}560}{1.5} + .5 + 3.6\right) \times \frac{15^2}{1.5^4}\right]$$
$$= 760.8 \text{ H. P. Ans.}$$

82. Effect of Size on Resistance.—By comparing the results of the two examples just given, it is seen that a reduction of 6 inches in the diameter of the pipe increases the resistances to such an extent as to require a pumping engine of 189.8 additional horsepower capacity. In the design of a pumping main, therefore, care and judgment should be exercised in securing a diameter of pipe as large as possible consistent with economy. The pipe should be laid in as straight a line as possible. A mean must be struck between the cost of pumping, on the one hand, and the cost of the pipe on the other.

83. Cost of Installing Pumping Machinery.—A steam plant for pumping purposes is made up of three main elements; namely, *boilers*, *engines*, and *pumps*. These may be had from manufacturers in endless variety and at various costs. In general, the cheaper the first cost the higher is the cost of maintenance; and it is foolish economy, when the amount of water to be pumped is large, and therefore the horsepower required is high, to install a cheap boiler and engine, which will burn a great deal of coal and require frequent renewals and repairs. Pumping engines delivering water to a reservoir or standpipe do not usually work continually, but work at their full load while they are in operation. This is the most efficient and economical method of running machinery of this sort. The engines are so designed that each day's necessary pumping can be completed within 8 or 10 hours,

this being, with the storage in the reservoir or standpipe, sufficient to provide for all domestic and manufacturing needs. In cases of fire, however, the rapid depletion of the reservoir should call the pumps at once into action, whatever the hour of the day or night. By pumping only 12 hours out of the 24, a night shift of attendants is avoided, which may be estimated (with a plant of moderate size where the engineer can do the firing) at $720 per year. This sum is the interest, at 5 per cent., on $14,400; so that it is economy to pay as much as $14,400 for an increased size of pumping plant in order to avoid night work. In reality, more than this amount could be expended, since the efficiency of the plant at night, when running light, is low, and the amount of coal burned is high for the amount of water pumped.

84. As a guide for making an approximate estimate of the cost of a pumping plant, the following figures are given:

Boilers may be estimated at $10 per horsepower for plain tubular boilers, and $14 for water-tube boilers. There seems

TABLE IV

COST OF VARIOUS TYPES OF ENGINE

Type of Engine	Cost per Horsepower	Steam Consumption, in Pounds, per Horsepower per Hour at Full Load
Simple slide valve	$ 9	33 to 45
Compound slide valve	14	25 to 30
Low-speed Corliss	12	24 to 30
Compound Corliss	20	18 to 21
Low-speed automatic	16	20 to 24
High-speed automatic	12	30 to 36

to be little difference in economy in the two types of boilers, and the advantage of the water-tube boiler lies chiefly in the greater safety from explosion, and in the rapid circulation, which insures quick firing. A non-condensing engine uses

more steam per horsepower than does a condensing engine, and much more than a compound-condensing engine. For ordinary tubular boilers, the cost of boiler installation per horsepower for these three types is about as follows: Compound-condensing engine, $10 per horsepower; condensing engine, $13.50 per horsepower; non-condensing engine, $16 per horsepower.

Engines of more than 100 horsepower will cost, when installed, about as shown in Table IV.

Piston pumps of ordinary make cost from $3 to $6 per horsepower, the former price being for non-condensing and the latter for condensing plants, both prices including the heaters.

The prices given do not include the cost of buildings and chimneys, which may be estimated at about $25 per horsepower, although this item may vary from this figure by $5 either way, according to the character of the structure erected and the locality where the plant is installed.

85. Cost of Running Pumping Machinery.—According to Frizell, the cost of running a steam plant of any kind may be divided into two parts; namely, the cost of the coal required, and the cost of attendance, lubricants, waste, repairs, and miscellaneous items. Where coal is cheap and the plant is economical in the use of it, the other items of expense will be much greater than the coal expense. But where little attention is given to the consumption of coal, and where the plant consists of a simple engine and a cheap pump, the cost of the coal will be in excess. The consumption of coal in any plant is a variable factor often depending to a great extent on the fireman. An average capacity for good boilers is an evaporation of 11 pounds of water per pound of coal, so that, by Table IV, a simple slide-valve engine using 44 pounds of steam per horsepower per hour would require 4 pounds of coal; whereas, a low-speed automatic engine using 22 pounds of steam per horsepower would require 2 pounds of coal per horsepower, or one-half the amount needed for the other engine. A

100-horsepower engine in the one case would consume about 5 tons per day, and the other $2\frac{1}{2}$ tons. Such values as these, however, while useful in making comparisons, are not equaled in actual operation, on account of fluctuations of loading, and general deterioration of the plant.

86. Efficiency of Engines.—The efficiency of engines varies both with the character of the engine and with the load. Large engines working with full load may have an efficiency of 90 per cent., but with half load the efficiency would be only about 80 per cent., and with quarter load but 65 per cent. Small engines may have an efficiency of 80 per cent. with full load, decreased to 70 per cent. at half load, and to 40 per cent. at quarter load.

87. Efficiency of Pumps.—The efficiency of pumps, especially of small ones, discharging up to 2,000,000 gallons per day, is low, probably not more than 50 per cent., partly on account of the friction in the cylinders, and the loss due to the continual change in momentum of the water; and partly because of the large losses from radiation, on account of the relatively large steam cylinders. With large pumps, under constant load, and designed especially for a particular duty, a greater efficiency may be obtained, and tests have shown it possible to secure an efficiency of 91 per cent. under the most favorable conditions.

88. Duty of Pumping Engines.—A common and useful method of stating the effectiveness of a pump and engine, or of a pumping engine in combination, is to express the number of foot-pounds of work done by 100 pounds of coal. This number is called the **duty** of the engine or plant. Thus, if 1,000,000 pounds of water is raised 100 feet by a pump with the consumption of 200 pounds of coal, the duty of the plant is 50,000,000. Small pumps working up to 1,000,000 gallons per day may have a duty of 10,000,000 to 20,000,000. Larger plants, if of the best class, may have a duty as high as 50,000,000. Special pumps of the largest size, designed for special work, and costing many thousand dollars, may have a duty as high

as 150,000,000; but such pumps serve to show rather what can be done under the best conditions than what may generally be expected.

89. Cost of Pumping Water.—The cost of pumping water is approximately as follows: In a general way, the cost of water may be estimated at a certain sum per million or per thousand gallons. It is found by experience that, in the best and largest plants, where the engines are of the most economical form, and where the plant is specially designed, it costs at the rate of about 5 cents for each million gallons lifted 1 foot. For smaller plants, the costs of lifting 1,000,000 gallons 1 foot are about as follows:

CAPACITY OF PLANT *Gallons per Day*	COST OF LIFTING *Cents*
10,000,000 or more	5
1,000,000	10
100,000	15

Intermediate quantities may be estimated at intermediate proportionate amounts.

WATER SUPPLY

(PART 3)

RESERVOIRS

CLASSIFICATION

1. Classes of Reservoirs.—Reservoirs may be divided into two main classes; namely, *storage reservoirs* and *distributing reservoirs.*

2. Storage reservoirs are required when, the average flow of the stream from which the supply is to be taken being greater than the average consumption, the minimum daily or monthly flow of the stream is less than the corresponding daily or monthly consumption. Under such conditions, the excess of water discharged by the stream in times of large flow is stored in a storage reservoir, to make up for the deficiency in times when the flow of the stream is not sufficient to meet the demands of consumption. If the average flow of the stream is less than the average consumption, no reservoir, of any size, can meet the demands of consumption. If the minimum stream flow is greater than the maximum consumption, no storage reservoir is needed. It is for conditions between these two extremes that a storage reservoir is necessary.

3. Distributing reservoirs, on the other hand, are intended to equalize the consumption throughout a single day. They are built, not on the stream, but at some point

on the pipe line. They may also serve the additional purposes of relieving the pipe line of excessive pressure, in the case of a gravity supply from a great elevation; and, in the case of a pumped supply, to give head enough to distribute the water by gravity. Distributing reservoirs may either be built in the ground, like storage reservoirs, or be entirely artificial and built of steel in the form of standpipes or elevated tanks.

STORAGE RESERVOIRS

4. Location of Storage Reservoirs.—The most suitable site for a storage reservoir is one that will store the maximum amount of water with the minimum cost of dam construction. Usually, some valley or bottom land, the sides of which approach each other at some point so as to form a natural entrance into the valley, is utilized for the purpose. There are other factors to be considered, however. The foundation on which the dam is to be built should be of rock, if possible, both to insure the stability of the dam and in order that, by carrying the dam into the rock, all danger of water leaking through under the dam may be avoided. The storage reservoir should also, if possible, be at such elevation that it insures a gravity supply, giving sufficient head for all purposes.

It is very rare that a location embracing all these features can be found. The skill and judgment of the engineer are exercised in selecting a site that, although it may not fulfil all the desired conditions, yet will most satisfactorily fulfil those that, under the circumstances, seem to be the most important.

5. Examination of the Ground.—In finally selecting a proper site for a storage reservoir on any stream, the first step is to make a reconnaissance of the part of the valley where appearances indicate that a good site will be found. The topographic sheets of the Geological Survey, in the United States, and the maps of the Ordnanoe Survey, in England, indicate where such a site may be expected, and to

which part of the valley further studies should be directed. In studying the valley, the necessary elevation of the water surface to give the required head must be kept in mind. The possible height of the dam must generally be approximated from the surrounding topography.

For such a reconnaissance, the engineer should be provided with a pocket compass, an aneroid barometer, a hand level, and a sketchbook or notebook. The aneroid is read along the bottom of the valley, at the foot of the side slopes, and on the tops of low ridges that may determine the height to which the water behind the dam can be raised. In order that these readings may be of any value, a stationary barometer kept in the vicinity of the proposed site should be read by an assistant at intervals of not more than 10 minutes. The variations of this stationary barometer, due to atmospheric changes, are used at the end of the day to correct the readings of the field instrument. The method of procedure is as follows:

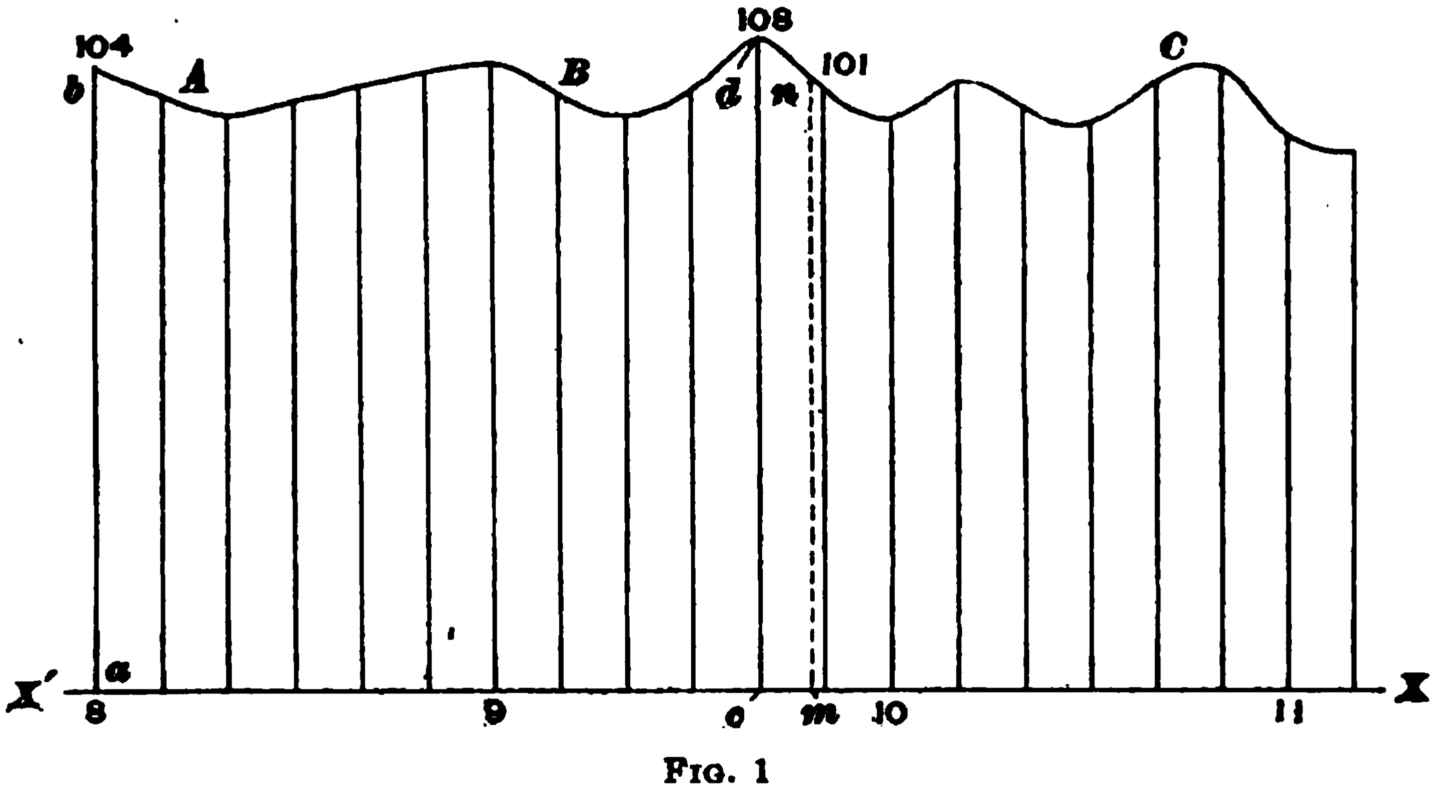


FIG. 1

On a base line $X'X$, Fig. 1, are laid off as abscissas the times at which the readings of the stationary barometer are taken. The figure represents only a part of the record, between 8 and 11:10 A. M.; each hour is divided into six equal parts, it being assumed that readings are taken every 10 minutes. The ordinates to the base $X'X$ represent barometric readings, diminished, for convenience, by any convenient

amount, as 1,000 feet. Thus, if the reading at 8 A. M. is 1,104, the ordinate *ab* is made, to any convenient scale, equal to 1,104 – 1,000, or 104. This is equivalent to taking the datum plane *X′X* as 1,000 feet above sea level. All elevations must, of course, be referred to the same datum. If, at 9:40, the barometer reads 1,108, the corresponding ordinate *cd* is made, to scale, equal to 1,108 – 1,000, or 108, as marked. At the end of the day, or of the period of observation, the extremities of the ordinates are connected by a curve *ABC*, from which intermediate elevations can be readily scaled off.

When the readings in the field are taken, the time of each reading should be carefully noted. From each reading, 1,000 should be subtracted to reduce the elevation to the adopted datum. Suppose that a reading of 1,118 was taken at 9:48. This reading, which will be recorded as 118, must be corrected by comparing it with the corresponding reading of the stationary barometer. Scaling off on *X′X*, 48 minutes after 9, and measuring the corresponding ordinate *mn*, it is found to measure 102.5. This shows that, between 8 and 9:48, the atmospheric conditions caused a fall of 104 – 102.5, or 1.5, feet, from the height at 8 A. M., which is taken as the standard. Therefore, any reading taken at that time must be increased by 1.5 feet, and, in the present case, the corrected reading, or true elevation, is $118 + 1.5 = 119.5$. If the diagram at any time shows a rise of the barometer above 104, the amount must be subtracted from the field readings taken at that time. All readings should be corrected as just explained.

6. The compass is used to make a rough survey of the valley, taking bearings of the directions of the several parts of the stream, of the sides of the valley, and of intermediate and side ridges. The distances may be estimated by pacing. In this way, a crude topographical map of one or more sites can be prepared, and that site selected which seems to deserve further consideration. Where maps are available, they will take the place of those prepared as here described;

otherwise, this approximate work will generally save both time and expense.

Besides the map of the immediate vicinity, the area of the watershed above the dam should be determined, in order that the drainage area and the amount of rainfall to be collected may be known.

7. Having selected the site that seems most desirable, further and more accurate surveys must be made. They may be made either with a plane table or by stadia measurements. A new map should then be drawn, showing, on a scale of about 400 feet to the inch, contours 5 feet apart, the location of all roads and buildings, the character of the vegetation and of the soil, the property lines and owners' names, together with any part of the area outside of the reservoir site that may be used in carrying off the surplus water from the reservoir in time of flood. A line should also be run and plotted between the reservoir and the city or town, so that the difference in elevation and the length of main may be known. At the site of the dam, borings, or test pits, must be made to ascertain the character of the soil on which the dam is to stand.

8. Amount of Storage.—The size of a storage reservoir depends on the relative quantities of the stream flow and the consumption, and is rather an uncertain quantity, since both of the factors involved are known only approximately. The following is an accepted method of estimating the necessary volume to be stored.

For the first step, there should be compiled, either from local records, or from United States Weather Bureau publications, the amount of rainfall each month, going back as many years as possible. The 12-month period giving the minimum rain is then selected as a basis. This is most easily done by platting the rainfall by months, the months being laid off as abscissas and the rainfalls as ordinates.

As the second step, there should be prepared a table of evaporation, giving the quantity of water lost each month by evaporation.

As the third step, there must be found, from some published records of stream gauging, the percentage of the rainfall that is found in the stream.

As the fourth step, a table must be prepared by months, showing the demands of consumption.

With these four tables, the quantity of the stream flow can be determined month by month from the rainfall, while the evaporation and consumption represent the demands on the stream. The sum of the differences between these two results for the successive months when the stream flow is less than the demand is the required storage.

EXAMPLE.—To find the storage necessary to furnish a uniform supply of 50,000,000 gallons per day, when the rainfall in inches, for the 12-month period of minimum rainfall, has been found to be as follows:

Oct. 2.96	Jan. 2.81	Apr. 1.85	July 2.68
Nov. 4.09	Feb. 3.87	May 4.19	Aug. 0.74
Dec. 2.30	Mar. 1.78	June 2.40	Sept. 1.52

The area of watershed is supposed to be 150 square miles.

SOLUTION.—In Fanning's "Water Supply" is a table of percentages of rainfall found on the Sudbury River, Massachusetts, and the values there given have been extended into the following table, which represents gaugings for a period of 16 consecutive years.

TABLE A

Percentage of Rainfall Available as Run-Off

Month	Rainfall	Run-Off	Per Cent.	Month	Rainfall	Run-Off	Per Cent.
Jan.	4.18	2.05	49.1	July	3.78	0.34	8.9
Feb.	4.06	3.18	78.2	Aug.	4.23	0.55	13.1
Mar.	4.58	5.02	109.6	Sept.	3.23	0.46	14.2
Apr.	3.32	3.62	109.1	Oct.	4.41	1.02	23.1
May	3.20	2.00	62.3	Nov.	4.11	1.62	39.5
June	2.98	0.87	29.1	Dec.	3.71	1.95	52.5

Multiplying the monthly rainfalls given in the example by the ratio or percentage for the corresponding month, the run-offs given in Table B are found.

TABLE B

Run-Offs for Different Months

Oct. 0.68	Jan. 1.38	Apr. 2.02	July 0.24
Nov. 1.62	Feb. 3.03	May 2.61	Aug. 0.10
Dec. 1.21	Mar. 1.95	June 0.70	Sept. 0.22

Now, 1 in. of rainfall on an area of 150 sq. mi. is expressed in gallons as follows:

$$\tfrac{1}{12} \times 150 \times 640 \times 43.560 \times 7.5 = 2{,}613{,}600{,}000 \text{ gal.}$$

Evaporation is assumed to take place from water surfaces only, and since the area of the storage reservoir is not yet known, this surface must be assumed. It is usual to assume this area as 3, 5, or 10 per cent. of the watershed area. Here, it will be taken as 5 per cent., a value to be revised later if necessary. Taking the evaporation values given in *Water Supply*, Part 1, for Boston, Table C is formed.

TABLE C

Inches of Evaporation per Month

Oct. 3.16	Jan. 0.96	Apr. 2.97	July 5.98
Nov. 2.25	Feb. 1.05	May 4.46	Aug. 5.50
Dec. 1.51	Mar. 1.70	June 5.54	Sept. 4.12

The number of gallons lost by the evaporation of 1 vertical inch is 2,613,600,000 × .05 = 130,680,000. Multiplying this quantity by the monthly evaporation values given in Table C, the demands of evaporation per month are as given in Table D.

TABLE D

Total Loss by Evaporation in Million Gallons

Oct. 412.95	Jan. 125.45	Apr. 388.12	July 781.47
Nov. 294.03	Feb. 137.21	May 582.83	Aug. 718.74
Dec. 197.33	Mar. 222.16	June 723.97	Sept. 538.40

From these data, Table E is prepared. The first column contains the product of 2,613.6 by the run-off, in inches, given in Table B. The second column contains the amount of evaporation given in Table D. The third column is the monthly consumption, which has an average value of 50,000,000 gal. per day, but varies, as shown, from month to month, as explained in *Water Supply*, Part 1. The fourth column is a deduction to be made from the amount of storage on account of percolation, or leakage, through the dam, or into the surrounding earth. This is a very uncertain quantity, depending on the character of the soil and the height of the dam. It is here assumed constant and equal to 1 in. of rain, or 2,613,600,000 gal. per year. The fifth and sixth columns are the differences between the first column and the sums of columns 2, 3, and 4, the fifth column showing when the difference is plus or when there is a surplus in the stream, and the sixth column showing when the difference is minus, or when there is a deficiency in the stream. All quantities are in millions of gallons.

9. Remarks.—Whenever gaugings of the stream on which a storage reservoir is to be built are available, they ought to be compared with the rainfall over the watershed

of that area, and those ratios used instead of those in Table A of the preceding example. Table E, however, has been carefully prepared, and is approximately correct for the

TABLE E

Collected Data (Million Gallons)

Month	Stream Flow Available for Storage	Monthly Evaporation	Monthly Consumption	Percolation	Surplus	Deficiency
October .	1,777.3	412.95	1,300	217.8		153.45
November	4,234.0	294.03	1,250	217.8	2,472.2	
December	3,162.5	197.33	1,420	217.8	1,327.4	
January .	3,606.8	125.45	1,500	217.8	1,763.5	
February	7,919.2	137.21	1,760	217.8	5,804.2	
March . .	5,096.5	222.16	1,410	217.8	3,246.5	
April . .	5,279.5	388.12	1,250	217.8	3,423.6	
May . . .	6,821.5	582.83	1,300	217.8	4.720.9	
June . . .	1,829.5	723.97	1,590	217.8		702.3
July . . .	627.26	781.47	1,750	217.8		2,122.0
August .	261.36	718.74	1,910	217.8		2,585.2
September	574.99	538.40	1,560	217.8		1,741.2
					22,758.3	7,304.2

NOTE.—The sum of the quantities in the sixth column is the deficiency in the stream flow for the months of June, July, August, September, and October, and is the storage supply required.

eastern half of the United States. In preparing column 3, the following percentages of the average monthly consumption may be used:

Jan. 87	Apr. 90	July 121	Oct. 101
Feb. 90	May 98	Aug. 115	Nov. 94
Mar. 87	June 150	Sept. 110	Dec. 92

10. Water-Storage Table.—Table I may be used as a brief method of determining the storage necessary to supply a definite quantity of water. It is taken from a table given by Desmond FitzGerald in the Transactions of the American Society of Civil Engineers, Vol. 27, page 267. It is based on studies of the yield of the Sudbury river throughout a

long period of years, and may be applied, with some degree of approximation, to the eastern half of the United States.

TABLE I

STORAGE IN GALLONS PER SQUARE MILE OF WATERSHED

Constant Daily Draft per Square Mile of Watershed Gallons	Water Surface 0 Per Cent. of Watershed	Water Surface 10 Per Cent. of Watershed	Water Surface 25 Per Cent. of Watershed
100,000	314,000	15,012,000	53,565,000
150,000	3,006,000	19,642,000	59,665,000
200,000	8,797,000	25,742,000	65,765,000
250,000	17,997,000	33,338,000	71,865,000
300,000	28,473,000	43,437,000	78,807,000
350,000	39,173,000	54,137,000	87,957,000
400,000	51,303,000	66,050,000	99,089,000
450,000	63,553,000	78,300,000	127,412,000
500,000	75,803,000	90,550,000	156,362,000
550,000	88,053,000	105,987,000	185,312,000
600,000	100,651,000	134,937,000	214,262,000
650,000	114,451,000	163,887,000	250,744,000
700,000	139,950,000	192,837,000	336,044,000
750,000	168,900,000	221,787,000	421,344,000

EXAMPLE.—Let it be required to determine the necessary storage capacity to meet a constant daily draft of 50,000,000 gallons from a watershed of 150 square miles, with 5 per cent. of water surface.

SOLUTION.— 50,000,000 gal. from 150 sq. mi. is 333,333 gal. per sq. mi. Interpolating between the values given for 300,000 and for 350,000, for 333,333, which is $\frac{2}{3}$ the difference, the value for 0 per cent. is 35,606,300 gal., and that for 10 per cent. is 50,570,300 gal. Therefore, for 5 per cent., the mean of these values is taken, which is 43,088,300, storage per sq. mi. The total storage is 43,088,300 × 150 = 6,463,245,000 gal. This is a smaller amount than was obtained in Art. 8, but an amount that Mr. FitzGerald claims will give safe storage for New England conditions.

EXAMPLES FOR PRACTICE

1. What is the necessary storage capacity to supply 108,000,000 gallons per day from a watershed of 200 square miles, with 5 per cent. of water surface? Ans. 94,251,300 gal.

2. What amount of storage is necessary to furnish a daily supply of 48,000,000 gallons from a watershed of 100 square miles, with 12 per cent. of water surface? Ans. 93,534,300 gal.

---

11. Height of Dam.—After the amount of water that the reservoir is to hold has been determined, it is necessary to compute the height to which the dam must be built in order to impound that amount of water, or, if the height of the dam is limited by the height of the sides of the valley, to determine whether the reservoir thus built will contain the water found to be necessary. On account of the effect on the quality of the water, it should not be necessary to draw the water from below a certain level, because the sediment that accumulates in the bottom of a reservoir is usually laden with impurities. It is, further, often the case that, on account of the head required to produce a certain pressure in the water pipes, only the water at the top is of use, and all the water stored below that level is useless.

When the height of the dam is limited by the height of the ridges enclosing the valley, the volume of the basin within the ridges is computed by the methods explained in *Hydrographic Surveying*. If the result is less than the required storage, another site must be selected, or two or more reservoirs must be built at various sites. If the height of the dam is not limited, or if the volume corresponding to the limiting height is greater than the required storage, the necessary height of dam is determined as follows:

A contour map of the basin, with a contour interval of 5 feet, is first prepared. A depth of, say, 10 feet above the base of the inner slope of the dam is decided on, below which no water is to be drawn. Starting with the contour at this depth, the volume between it and the next higher, which will be called the *second contour*, is computed by the method of average end areas; then, the volume between the second

contour and the next higher, which will be called the *third contour*, is computed by the same method, and the result added to the volume first determined; and so on until a volume is obtained equal to or slightly in excess of the required storage volume. The calculation of the whole volume is then repeated, using the prismoidal formula. If the result is smaller than the required storage volume, one contour is added, the volume between it and the last one previously used being computed by the average end-area method. If this volume is not sufficient to make up the deficiency, another contour is added, and so on. If the result obtained by the prismoidal formula is much in excess of the required storage, a similar operation to that just described is gone through, except that the contours are diminished instead of increased. The elevation of the final top contour is the elevation of the spillway of the dam. That contour is called the **flow line** of the reservoir.

EXAMPLE.—It is required to store 32,164,000 gallons of water. The water below contour 50 is not to be used. The area included by contours *50*, *55*, *60*, *65*, etc., denoted by A_{50}, A_{55}, etc., are as follows:

A_{50} = 126,540 sq. ft.	A_{70} = 145,900 sq. ft.
A_{55} = 134,300 sq. ft.	A_{75} = 160,000 sq. ft.
A_{60} = 156,000 sq. ft.	A_{80} = 162,000 sq. ft.
A_{65} = 125,500 sq. ft.	A_{85} = 157,600 sq. ft.

To determine the elevation of the spillway of the dam.

SOLUTION.—The required volume, in cu. ft., is 32,164,000 ÷ 7.48 = 4,300,000. The volume included between the first two contours is $\frac{5}{2}$(126,540 + 134,300) = 652,100 cu. ft. The volume included between the second and third contours is $\frac{5}{2}$(134,300 + 156,000) = 725,750 cu. ft. Adding this volume to that previously obtained, we have 1,377,850. It is seen that this is not nearly enough. The volume included between contours *3* and *4* is $\frac{5}{2}$(156,000 + 125,500) = 703,750 cu. ft.; that between contours *4* and *5* is $\frac{5}{2}$(125,500 + 145,900) = 678,500 cu. ft.; that between contours *5* and *6* is $\frac{5}{2}$(145,900 + 160,000) = 764,750 cu. ft. The sum of all the volumes obtained is 3,524,850 cu. ft. This is not quite enough. The volume included between contours *6* and *7* is $\frac{5}{2}$(160,000 + 162,000) = 805,000 cu. ft. 3,524,850 + 805,000 = 4,329,-850 cu. ft. The entire volume is now calculated by the prismoidal formula, as given in *Hydrographic Surveying;* namely,

$$V = \frac{h}{3}(A_0 + 4\,\Sigma A_1 + 2\,\Sigma A_2 + A_n)$$

Substituting the given values in this formula,

$$V = \frac{5}{3}[126,540 + 4(134,300 + 125,500 + 160,000) + 2(156,000 + 145,900) + 162,000] = 4,285,900 \text{ cu. ft.}$$

This is less than the required volume; therefore, another contour must be added. The volume included between contours *80* and *85* is $\frac{5}{2}(162,000 + 157,600) = 799,000$ cu. ft. The total volume is, therefore, $4,285,900 + 799,000 = 5,084,900$ cu. ft., and the storage capacity is $5,084,900 \times 7.48 = 38,035,100$ gal. The elevation of the spillway is, therefore, 85 ft. Ans.

12. Running the Flow Line.—A flow line must often be run on the ground, either to determine the area of a proposed water level or to mark out on the ground the land that will be flooded by a reservoir the land for which must be purchased. This operation is the same as running a contour line of given elevation, and is performed as described in *Topographic Surveying*, under the heading Direct Location of Contours. Having run this line, the area bounded by it should be carefully computed by the methods explained in *Trigonometry*, Part 2, *Compass Surveying*, Part 2, and *Transit Surveying*, Part 2.

13. Surveying the Site for the Dam.—After the operations described in the foregoing articles, a careful survey of the site of the proposed dam is made, to establish

FIG. 2

with more precision than was previously necessary its exact location and dimensions. Cross-sections across the valley are taken not only at the place that appears to the eye to be the most favorable, but also at other places both above and

below. These cross-sections are used in making a contour map, similar to that shown in Fig. 2, of the territory included within the site of the dam; and on this map a paper location of the dam can be made more understandingly than is possible by a mere inspection of the ground. This paper location is then staked out in the actual position the dam is to occupy, and carefully examined to see if any small changes can be made whereby the cost of construction may be reduced. The design and construction of dams are treated in the Section on *Dams*.

14. Preparing the Bottom of the Reservoir.—The natural and preliminary method of preparing the reservoir site is first of all to cut off the growing timber, if there is any, and to gather up and burn all the brush and rubbish remaining. Beyond this, what is done depends on the site itself and on the care expended on the quality of the water to be stored. Much money has been expended by some cities for the removal of top soil from reservoirs, in order to improve the quality of the water by ridding it of organic matter. It is doubtful whether such extreme precautions are of much value.

If any part of the area has been occupied by human beings, it should be thoroughly cleaned; the contents of privies should be removed, and all polluted soil dug up. Polluted areas of large size may be covered with sand or gravel if this is more convenient than cleaning them out.

15. Stream Compensation.—Every owner that has property on the bank of a non-navigable stream has certain rights in the stream, and one of those rights is to have the stream flow by his property in its usual and ordinary volume. If an individual or a city builds a dam across a stream and thereby impounds water that would naturally flow down the stream, any owner on the bank of the stream who may need the water for any purpose has a cause of grievance, and may apply to the courts for compensation. There are two methods of arranging this matter when a city plans a storage reservoir on a stream where objection is anticipated. One

method is to buy up all the rights of the owners down stream. Another method is to allow a certain amount of water always to run in the stream; the exact amount must be agreed on with the lower owners, or may be decided by the courts. The general principle is that all the water that generally can be used for power, stock, etc. must be allowed to flow, and only that excess be stored that cannot be used by the bank owners. This usually means that the storage must come from the large floods, and that the ordinary flow of the stream must be left undiminished.

16. Outlet Pipes.—For taking water from a storage reservoir by means of outlet pipes, certain requisites must be kept in mind. The end of the outlet pipe must be so arranged that water can be taken into the pipe at different levels, both to allow for fluctuations of level of the water, and so that the purest water may be taken, since the quality of water in a reservoir varies with the depth at different seasons of the year. Again, there must be provided proper gates or valves for turning on or off the water, for changing the level at which the water is drawn, and for drawing off all the water when the reservoir is to be cleaned. Since valves are uncertain and likely to get out of order, it is desirable, if possible, to have them where they may be examined and, if necessary, repaired. These requirements have caused the adoption of a tower, built up from the bottom of the reservoir and some distance back from the dam, or else a well built in the masonry of the dam itself, into which the outlet pipe is led; on the sides of the tower or well as many gates as desired are fastened, all of which can be readily inspected by draining out the water and climbing down a ladder provided for the purpose. Further details on this subject are given in *Dams*.

DISTRIBUTING RESERVOIRS

17. Location.—The location of a distributing reservoir depends largely on the topography of the surrounding country. The reservoir should be located as near as possible to the center of consumption, and at a sufficient elevation to give enough pressure for all domestic purposes. If no natural site of proper elevation exists, a steel tank or tower may be built to take the place of the reservoir. The reservoir would naturally be built between the storage reservoir or source of supply and the center of the city, although topography often interferes with this arrangement. For example, at Cohoes, New York, a city built on the banks of the Mohawk River, the water supply is pumped from the river to a distributing reservoir beyond the city, from which it flows back into the distributing system. Harrisburg, Pennsylvania, situated on the Susquehanna River, pumps its water supply across the city, a distance of 2 miles, into a distributing reservoir, whence it runs back to the city. In a large city, two or more reservoirs are desirable, conveniently distributed over the area supplied.

18. Advantages of a Distributing Reservoir.—In a gravity system, where water is delivered to a town from a

FIG. 3

high storage reservoir, it is often advantageous to insert one or more distributing reservoirs between the main reservoir and the town. In the first place, this lessens the danger of the service being interrupted on account of breakage in the

pipe between the main reservoir and the town; for, if a breakage occurs between the storage and the distributing reservoirs, the latter may supply the town while repairs are made. Another advantage is the economy that may be effected in the size of mains.

This point will be better understood by an illustration. Suppose that a city *C*, Fig. 3, is to be supplied from a storage reservoir *A*, and that it is possible to build a distributing reservoir at some intermediate point *B*. If the main is laid directly from *A* to *C*, it must be designed to discharge at the rate of the maximum hourly consumption; this will require a pipe of large diameter. The same amount of water, however, may in some cases be discharged from *A* into *B* with a much smaller pipe, and distributed from *B* to *C* through a larger pipe. In order to determine whether the construction of the reservoir at *B* is economical, it is necessary to find the price of the construction with and without that reservoir, and see which is the smaller.

19. In connection with a pumping plant, there may be some economy in the construction of a distributing reservoir aside from the insurance it gives against break-downs. If there is no reservoir, the pumps must have capacity enough to deliver the maximum amount of water, although they will work at that rate for only about 6 hours out of the 24. For 18 hours out of the day, then, they will work at less than their designed load, and therefore at a reduced efficiency. This loss may be more than enough to pay for the cost of a reservoir. In small cities, the maximum demands come from fires, and the pumps must be able to meet those demands.

20. Capacity of Distributing Reservoirs.—The capacity of a distributing reservoir depends chiefly on the location and estimated cost of the reservoir, and on the size of the city or town to be supplied. Where the city is built on level ground, and the only site for a reservoir is several miles away, so that the saving in the size of the pumping plant has to be offset by the cost of a long pumping main to the reservoir and by the cost of the return main, it is

probable that the cost of the reservoir and pipes will be too great to justify the construction; and then it is preferable to have a standpipe or elevated tank nearer the pumps. According to the experience of different cities, it requires from 2 to 4 hours' storage to allow a uniform flow of water from the pumps. In the case of a fire in a small city of 5,000, the demands of a large fire require a full day's supply, and a distributing reservoir intended to meet this demand will need to be large enough to hold that amount. A city of 100,000, on the other hand, will require about 6 hours' supply with no pumping, and a distributing reservoir would need to have that capacity, if no reserve capacity were given to the pumps.

In some cities, it is not economical to run the pumps at night. Then, the capacity must be such that there will be water enough to last from the end of one pumping to the beginning of the next, whatever that time may be. There is a great insurance against water famine in a city that has a distributing reservoir holding a week's supply. In such a case, the pumps may break down and be repaired without interfering with the supply. The fluctuations of seasons are taken care of by the reservoir, and a smaller pumping plant may be installed. It may indeed be considered possible, not merely to cut down the reserve capacity of the pumps, but also to eliminate the duplicate pumps that must ordinarily be installed.

In general, the question of size may be said to depend largely on the reliability of the source of supply. Where that source is sure to deliver the water regularly and constantly, the reservoir need be only large enough to equalize the daily fluctuations.

21. Number of Pipe Connections.—In a distributing reservoir, there are usually four pipes that enter the reservoir walls; namely, the main supply pipe, the main discharge pipe, the overflow pipe, and the drain pipe. The pipes must all have valves. In some reservoirs, all the pipes pass through one corner, where the valves are all placed in a

gate chamber provided for the purpose. The arrangements are similar to those described in *Dams*. In order that there may be no stagnation of the water in parts of the reservoir, the inlet pipe and the outlet pipe should be at opposite sides of the reservoir, and it reduces the possibility of stagnation still further to have the inlet near the surface and the outlet near the bottom of the opposite side. Fig. 4 shows a section of the distributing reservoir at Birmingham, Alabama, and the position of the four pipes mentioned.

The force-main from the pumps divides at A, a 20-inch pipe leading into R_1 and a 12-inch pipe into R_2. The reser-

FIG. 4

voir is in duplicate on account of necessary increased capacity. At B, a by-pass branches and extends around the reservoirs so that if at any time the reservoirs are both out of order, water may be forced directly into the mains. C_1 is the overflow from R_1; there is a similar overflow from R_2. D_1 is the drain pipe leading out from R_1; the drain from R_2 is not shown. E_2 is the main supply pipe leading from R_2, the main from R_1 coming out of R_1 at the same point as D_1, the drain. Gates are placed as shown on the various pipes.

STANDPIPES

LOCATION AND CAPACITY OF STANDPIPES

22. Location.—There are three factors to be considered in the location of a standpipe, and the designer must reconcile, as far as possible, the conflicting conditions involved in these three factors. Rarely does the topography of the town allow all the conditions to be satisfied at the same time, and it is always a question how far the advantages of one site from the standpoint of one of these conditions may be allowed to outweigh the disadvantages from the standpoint of another.

1. The standpipe should stand on the highest ground available. A rise of ground amounting to only 10 feet saves that much in the height of the standpipe. The value of a given elevation for the base of the pipe may be estimated as follows: Determine, according to the principles to be explained presently, the proper size of the tank; with the diameter and thickness, compute the weight of steel required for each foot of height, and, taking the cost of the pipe at 4 cents per pound erected, compute the total cost of the pipe. The cost thus found may be used to compare with the saving made by placing the standpipe at some lower elevation.

2. The standpipe should be located as near the center of distribution as practicable, so that the cost of the mains to and from the standpipe may be as low as possible, and that the loss of head in these mains may be reduced to a minimum. It would be possible to put the standpipe on a hill 20 feet high, and to have this hill so far from the part of the city where the large consumption occurs that the 20 feet is all lost in friction; so that a standpipe at a lower elevation, but nearer the center of consumption, would give an equal or greater pressure in the mains. The only way of comparing

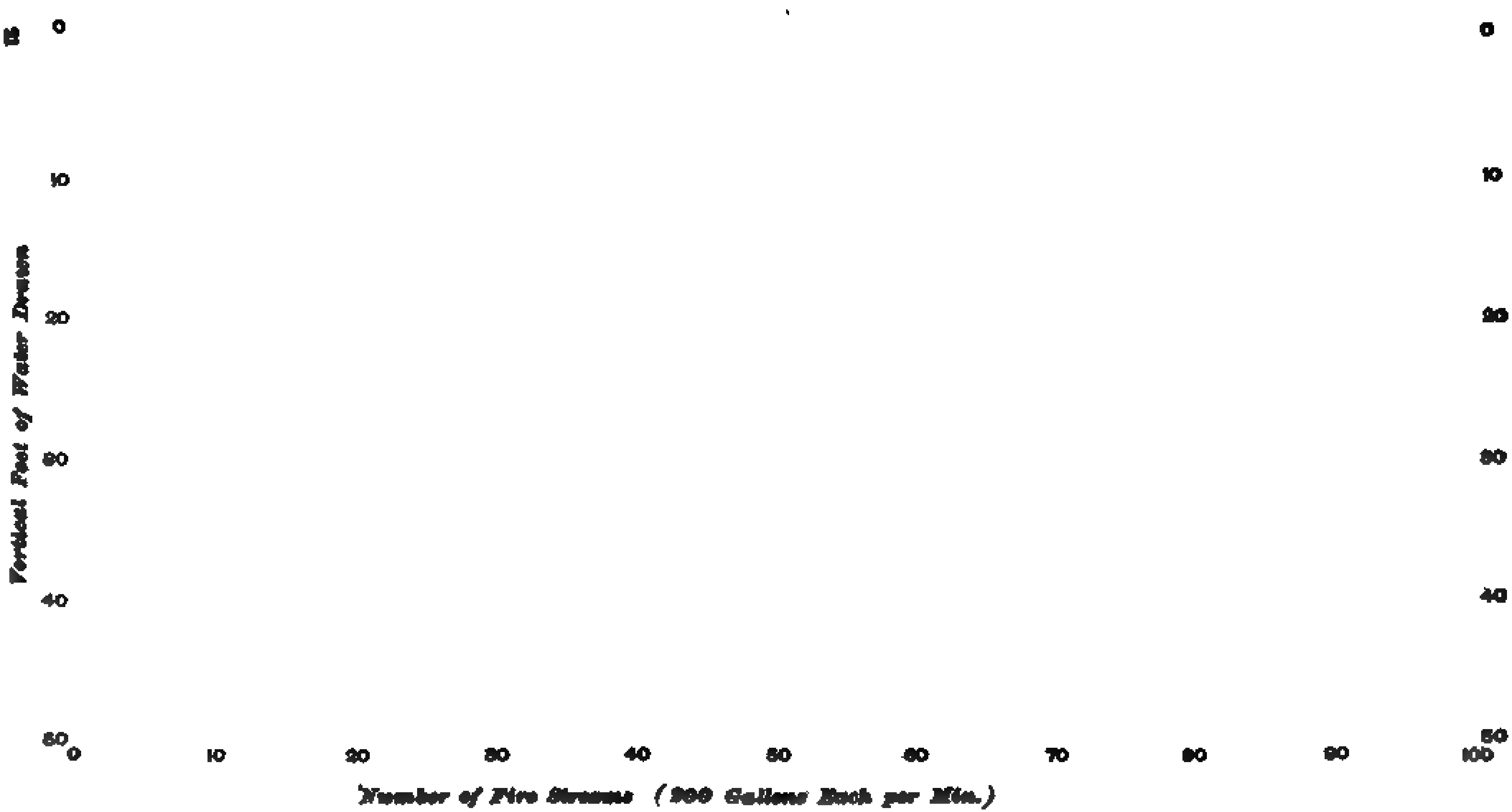
the relative advantages of various proposed locations is to compute the sizes and cost of the water pipes as well as of the standpipe for the different locations, and adopt the location that involves the least cost.

3. The standpipe should be located as near to the pumps as possible, in order to diminish the friction in the force mains and consequently the power required to work the pumps.

23. Height.—The height of a standpipe depends on the pressure required in the mains. The pressure necessary for domestic service is the pressure required to overcome friction in the pipes and deliver the water at the highest fixture in the town with a pressure sufficient to make the water run freely from the faucets; this pressure should not be less than 15 pounds per square inch.

24. Diameter.—The diameter of a standpipe depends first on whether it is intended to have the standpipe act as a distributing reservoir or simply as a pressure regulator. If economy is a strong factor, the pressure function of the pipe will be alone considered, and a 6-inch or 8-inch wrought-iron pipe running up to the proper height answers the purpose. Such a pipe is connected to the main pipe leading from the pumping station, and is placed as near to the pumps as possible; it may be attached to the side of the pumping building. It implies a pumping plant of a capacity sufficient to meet all demands for water, even at times of greatest consumption, especially at times of fires, and serves the sole but useful purpose of relieving the pipe system of shocks from the pumps. This kind of pipe gives no storage whatever, and, with the decrease in the employment of direct pumping systems, is going out of use.

25. It is advisable to provide a certain storage capacity. That capacity depends on the excess of consumption at times of fire, and on the amount of water used at night when the pumps are stopped. As has been pointed out, the amount of water used for fires in small cities is largely in excess of the normal consumption, and if storage is provided for the



Vertical Feet of Water Drawn
Number of Fire Streams (250 Gallons Each per Min.)
0
10
20
30
40
50
60
70
80
90
100

Fig. 5

proper number of fire-streams, the pump capacity can be much reduced. The question, then, is to determine the amount of water that must be stored for this purpose, and then to compare the extra cost of the storage tank thus designed with the extra cost of the larger pumping plant that is required if no storage is provided for.

Fig. 5, taken from the publications of the New England Water Works Association, shows graphically the depths to which different numbers of fire-streams will deplete, per hour, standpipes of different diameters. For example, by the table given in *Water Supply*, Part 1, a city of 10,000 persons requires nine fire-streams of 250 gallons per minute each. Taking the point on the horizontal axis between 8 and 10, and following up vertically, the diagram shows that a 20-foot tank would be drawn down at the rate of about 45 feet per hour; a 25-foot tank, at the rate of 30 feet per hour; a 30-foot tank, at the rate of 22 feet per hour; and so on. A city of 50,000 inhabitants would require 20 fire-streams, and a 40-foot tank would be drawn down at the rate of 26 feet per hour. In this diagram, no account is taken of the delivery of the pumps, which would continue during a fire, nor of the domestic consumption, which also would continue.

Fig. 6, taken from the same publications, shows the depths that standpipes of different sizes would be drawn down during the interval (about 12 hours) when pumps are not running. For example, a city of 30,000 inhabitants with a standpipe 50 feet in diameter would, as seen by following the horizontal line across the diagram from the 30 mark (population) on the right-hand side to its intersection with the included line marked 50, draw down the water level 31 feet, as shown at the bottom line of the diagram directly below the above point of intersection. It is, then, a question to be decided by comparison of costs, including both original and maintenance charges, whether it is cheaper to pump all night or to provide the capacity of tank shown by the diagrams.

26. The capacity of the tank is determined by due consideration of the items mentioned. It is to be borne in

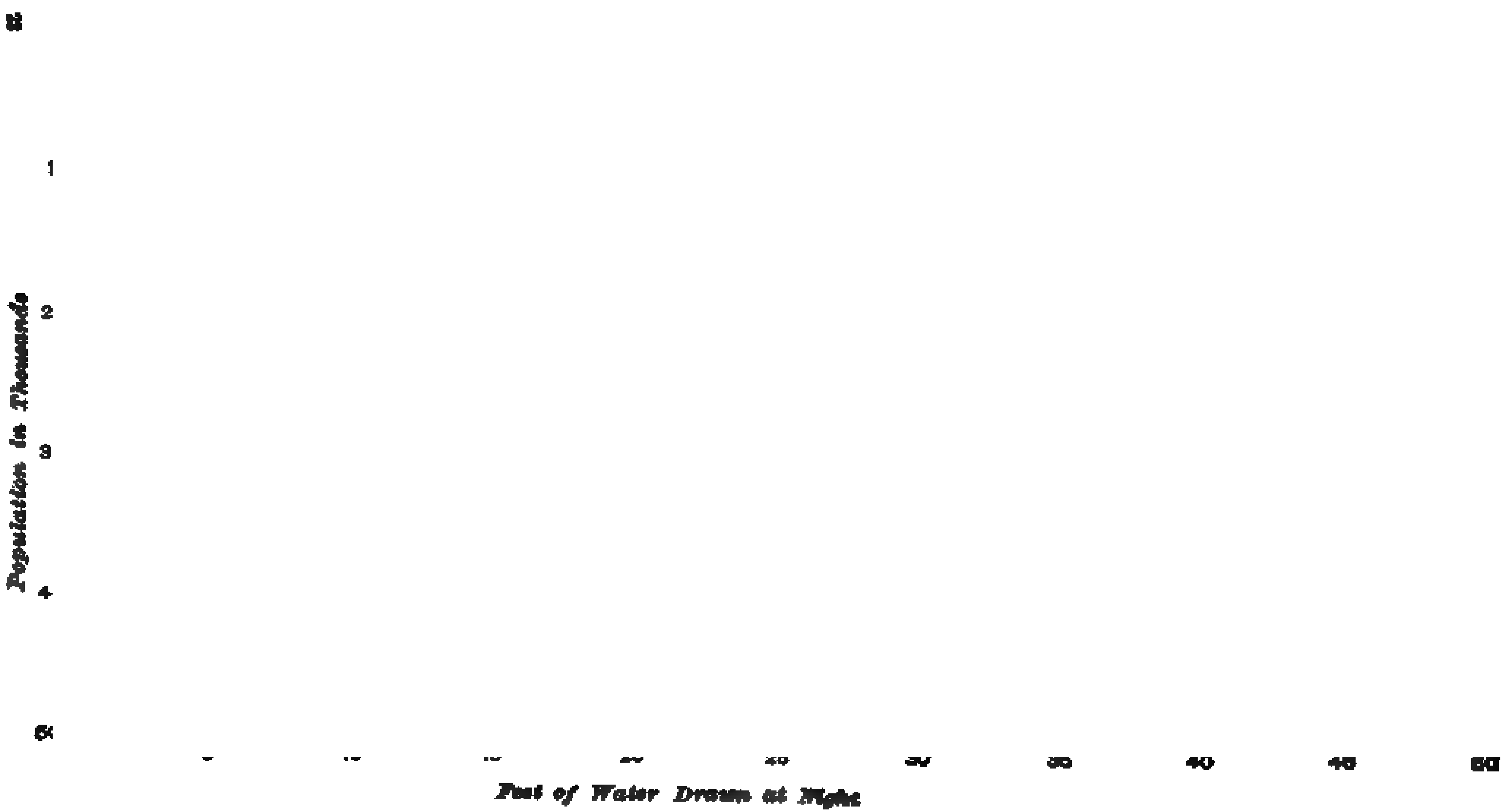

Population in Thousands
1
2
3
4
35
40
45
50
Feet of Water Drawn at Night

Fig. 6

mind that by the capacity of the tank is meant the volume between high-water and low-water levels of the tank. There is always, near the top of the tank, an overflow pipe that fixes the higher level. The lower level depends on the requirements of the distribution system, and is not absolutely fixed, since, when but little water is used, the same level gives a greater pressure than at other times when more water is used, as at times of fire. If the standpipe is on a hill, the lower level may be at the surface of the ground; in a flat district, it may be halfway up the pipe. In any case, the level necessary for a proper pressure for fires must be determined, and the capacity of the tank is the volume between this level and the top of the tank. Mr. Coffin, an eminent engineer, thinks that the permissible fluctuation should not exceed 50 feet.

27. It has been found by experience that a standpipe intended to stand alone, that is, not guyed or fastened to some building or chimney, should not have a height exceeding ten times its diameter, and preferably not more than eight times its diameter.

28. Thickness of Shell.—It was shown in *Strength of Materials* that, if t and d are, respectively, the thickness and diameter of a cylinder subjected to an internal pressure of intensity p, and if s is the working tensile strength of the material, then the highest value of p that will not subject the cylinder to a higher stress than the allowable working stress is given by the following formula:

$$p = \frac{2st}{d}$$

From this formula follows

$$t = \frac{p}{2s}d \qquad (a)$$

In the case of a standpipe, p is the pressure of the water, which, at any depth h (feet) below the surface, is $62.5h$ pounds per square foot, or $\frac{62.5}{144} \times h$ pounds per square inch (see *Hydrostatics*). The pipe is usually made of steel, with

a working stress of 15,000 pounds per square inch. To provide for riveting, however, only 60 per cent. of this value, or 9,000 pounds per square inch, is allowed. Making, in equation (a), $p = \frac{62.5\,h}{144}$, and $s = 9{,}000$, there results

$$t = \frac{62.5\,h}{2 \times 144 \times 9{,}000} \times d = .00002411\,h\,d \qquad (1)$$

In this formula, d and t are both in inches, and h in feet. If d is expressed in feet, the formula becomes, practically,

$$t \text{ (inches)} = .00029\,h\,d \qquad (2)$$

Example 1.—A standpipe 40 feet in diameter with plates all $\frac{1}{2}$ inch thick may be built on a hill, $\frac{1}{2}$ mile out of town, and be 60 feet high; or it may be built in the city and be 100 feet high. The water pipe is 8 inches in diameter, weighs 75 pounds per foot, and costs 2 cents per pound, laid. The weight of the steel is .278 pound per cubic inch and costs 4 cents per pound. To determine the cheaper location.

Solution.—Let h represent the height of the tank, and d represent the inside diameter of the tank, both in inches. Then, the outside diameter is $d + 1$. The volume of the steel in the standpipe is

$$.7854\,h\,[(d+1)^2 - d^2] = .7854\,h\,(2\,d + 1)$$

The cost of the tank 100 ft. high is, then,

$$.7854 \times 100 \times 12\,(2 \times 40 \times 12 + 1) \times .278 \times .04 = \$10{,}071$$

The cost of the tank 60 ft. high is

$$\frac{10{,}071 \times 60}{100} = \$6{,}042.6$$

The length of the water pipe is $\frac{5{,}280}{2} = 2{,}640$ ft., and the cost is $2{,}640 \times 75 \times .02 = \$3{,}960$. The total cost of the standpipe $\frac{1}{2}$ mi. out of town is $\$6{,}042.6 + \$3{,}960 = \$10{,}002.6$. Therefore, this location is cheaper by $\$10{,}071 - \$10{,}002.6$, or $68.40. Ans.

Example 2.—If the standpipe in the preceding example is placed $\frac{1}{2}$ mile out of town, what is the height required to overcome the losses due to friction, assuming the discharge to be 2 cubic feet per second at the time of maximum consumption?

Solution.—From Table IV, *Hydraulics*, Part 2, the value of $\frac{h}{l}$ for an 8-in. pipe discharging 2 cu. ft. per sec. is found to be about .017. Therefore, $h = .017 \times 2{,}640 = 45$ ft., nearly. Ans.

Example 3.—To determine the thickness of plate to be used in constructing a standpipe 50 feet in diameter and 100 feet high.

SOLUTION.—To apply formula 1, we have $h = 100$ and $d = 50 \times 12 = 600$. Substituting in the formula,

$$t = .00002411 \times 100 \times 600 = 1.45 \text{ in. Ans.}$$

EXAMPLES FOR PRACTICE

1. To what depth in 1 hour would a standpipe: (*a*) 50 feet in diameter, and (*b*) 100 feet in diameter be lowered by thirty fire-streams?

Ans. $\begin{cases} (a) & 24.5 \text{ ft.} \\ (b) & 6.5 \text{ ft.} \end{cases}$

2. How many feet is the water lowered during the night, in a standpipe 60 feet in diameter, the population being 30,000? Ans. 22 ft.

3. What should be the thickness of a standpipe 30 feet in diameter and 100 feet high? Ans. .87 in.

4. If the standpipe in the preceding example is 1 mile out of town, and the distributing main is 10 inches in diameter, what is the height required to overcome friction, the discharge being 3 cubic feet per second? Ans. 66 ft., nearly

---

STABILITY OF A STANDPIPE

29. Wind Pressure.—The pressure of the wind is a rather uncertain quantity. In the design of standpipes, it is customary to assume a wind pressure of 30 pounds per square foot, acting on a surface equal to the projection of the pipe on a vertical plane; that is, on a rectangular surface whose height is that of the pipe, and whose width is the diameter of the pipe. Thus, the maximum wind pressure on a pipe 20 feet in diameter and 60 feet high is $(60 \times 20) \times 30 = 36{,}000$ pounds.

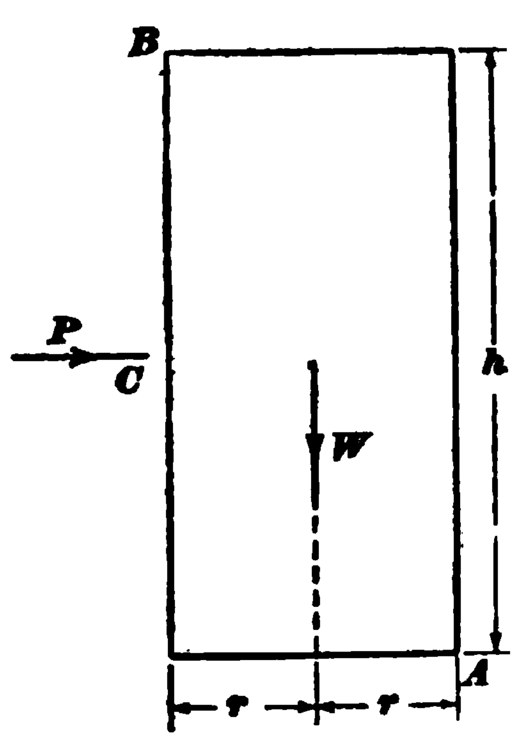


FIG. 7

30. Stability.—The resultant wind pressure P, Fig. 7, is assumed to act horizontally at the center C of the pipe AB. The tendency of that pressure is to upturn the pipe about the edge A. This tendency must be balanced by the weight of the pipe and the resistance of anchor bolts and guys provided for the purpose. In order

that the weight W of the pipe alone may resist the overturning tendency of the wind pressure, the moment of W about A must be at least equal to the moment of P; that is, Wr must not be less than $\frac{Ph}{2}$; if the former of these moments is less than the latter, the pipe must be stayed by anchor bolts or guys, as will be described further on.

EXAMPLE.—A standpipe 120 feet high and 20 feet in diameter, weighs about 150,000 pounds. Assuming the wind pressure to be 30 pounds per square foot, will it be necessary for the standpipe to be stayed?

SOLUTION.—The total wind pressure is $120 \times 20 \times 30$, or 72,000 lb. The overturning moment is $72{,}000 \times \frac{120}{2}$, or 4,320,000, ft.-lb. The moment of the weight about the same point is $150{,}000 \times 10 = 1{,}500{,}000$ ft.-lb., which is less than the moment of the wind pressure. Therefore, the standpipe must be stayed. Ans.

DESIGN AND CONSTRUCTION OF STANDPIPES

PLATES AND RIVETING

31. Materials Used.—Until recently, standpipes were made of wrought iron. At present they are almost invariably made of medium steel. The steel used for this purpose may be either open-hearth or Bessemer. Engineers differ as to which of these two is the better. Bessemer steel is the cheaper of the two, but does not seem to be so homogeneous as open-hearth steel, and, when used, should be very carefully inspected and tested.

In the market, the following grades of medium steel are recognized: *tank steel*, *shell steel*, and *flange steel*.

Tank steel is the cheapest and the poorest; it is made from inferior stock; it is high in phosphorus, and therefore brittle; although its tensile strength is high, it is deficient in homogeneity, and should not be used for standpipes. Some of the failures of standpipes are undoubtedly due to the use of this inferior grade of steel.

Shell steel is made more carefully and of better stock; it is more uniform, costs a little more, and does not have so

high a tensile strength as tank steel; it is more ductile, however. Shell steel is used in the manufacture of boilers, and many standpipes in use for many years have been made from it.

Flange steel has a uniform composition, has high ductility, and a comparatively low tensile strength. Although it costs more per pound than shell steel, and has a lower tensile strength, its homogeneity and ductility make it especially adapted to standpipe construction.

32. Thickness of Plates.—The formula by which the thickness of the plates is to be determined has been already given in Art. 28. Practically, certain limitations are placed on the application of this formula. The least thickness allowed is $\frac{1}{4}$ inch, thinner plates being too flexible. Plates thicker than $1\frac{1}{8}$ inches should not be allowed, since they are too thick to be properly riveted together. By the use of the formula, Table II has been prepared, which gives the proper thickness in terms of the product $h\,d$, both h and d being in feet.

TABLE II

THICKNESSES OF STANDPIPE FOR VARIOUS DEPTHS AND DIAMETERS

(*h = height, in feet; d = diameter, in feet; t = thickness, in inches*)

$h\,d$	t	$h\,d$	t	$h\,d$	t	$h\,d$	t
0– 600	$\frac{1}{4}$	1,600–1,800	$\frac{9}{16}$	2,600–2,800	$\frac{7}{8}$	3,450–3,600	$1\frac{3}{16}$
600– 900	$\frac{5}{16}$	1,800–2,000	$\frac{5}{8}$	2,800–2,900	$\frac{15}{16}$	3,600–3,750	$1\frac{1}{4}$
900–1,200	$\frac{3}{8}$	2,000–2,200	$\frac{11}{16}$	2,900–3,100	1	3,750–3,900	$1\frac{5}{16}$
1,200–1,400	$\frac{7}{16}$	2,200–2,400	$\frac{3}{4}$	3,100–3,300	$1\frac{1}{16}$	3,900–4,050	$1\frac{3}{8}$
1,400–1,600	$\frac{1}{2}$	2,400–2,600	$\frac{13}{16}$	3,300–3,450	$1\frac{1}{8}$		

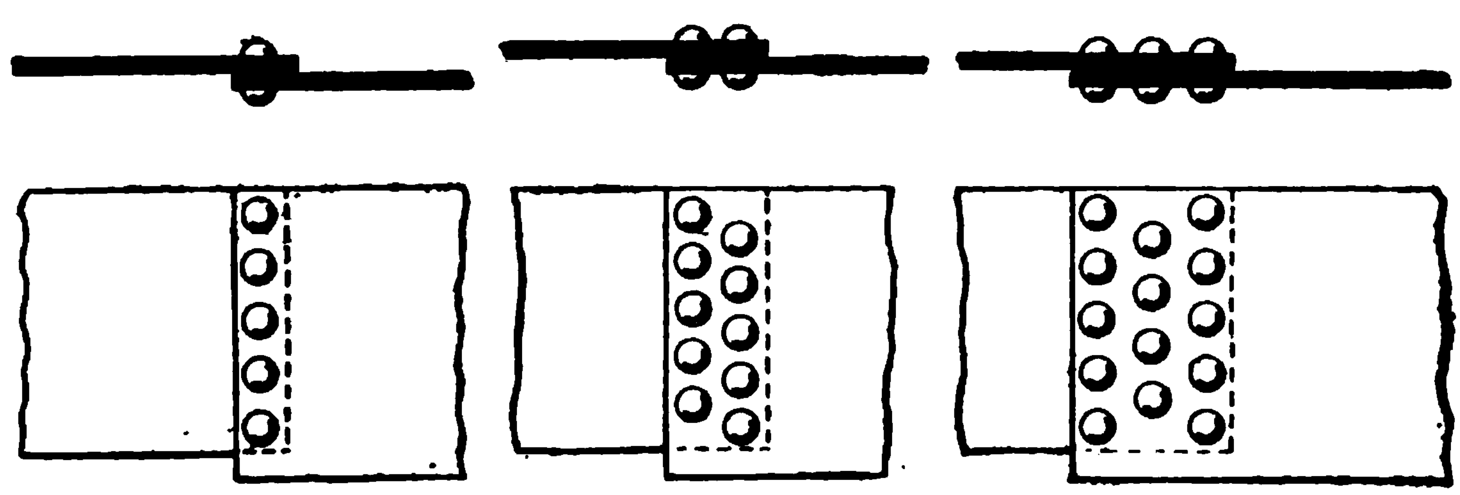
If plates thicker than $1\frac{1}{8}$ inches are required, it is better to reduce the diameter, or the height of the structure, if possible, or even to build two standpipes. It is also possible to substitute for the standpipe an elevated tank, in which the stresses against the plates are not so great. This subject will be fully treated further on.

33. Joints Between the Plates.—The shell of a standpipe is composed of a number of rings about 5 feet high, one above the other, the lower edge of each ring being riveted to the upper edge of the ring just below it by means of a **horizontal circular joint.** The stress on a horizontal joint is due to the weight of the standpipe above that joint, and to the bending moment caused by the wind pressure.

The plates that form each ring are about 8 feet in length, and are connected to each other by means of **vertical joints.** The stress on a vertical joint is due to the pressure of the water. The vertical joints form the most important part of a standpipe, since it is at them that the pipe is most likely to fail.

There are two kinds of riveted joints; namely, *lap joints* and *butt joints.* Riveted joints should be so designed that the strength of the rivets is about equal to that of the net section of the plate.

34. Lap Joints.—In a **lap joint,** the edge of one plate is placed over the edge of the other, so that the two plates will overlap, and the rivets are driven through the overlapping parts of the two plates. Fig. 8 shows three lap joints:



(a) (b) (c)

FIG. 8

in (*a*) there is one row of rivets, and the joint is said to be **single-riveted;** in (*b*) there are two rows of rivets, and the joint is said to be **double-riveted;** in (*c*) there are three rows of rivets, and the joint is said to be **triple-riveted.**

35. Single-Riveted Lap Joints.—Suppose that in Fig. 8 (*a*) the rivets are $\frac{3}{4}$, or 0.75, inch in diameter (0.4418 square inch in cross-section), and that each plate is $\frac{5}{16}$, or .3125, inch thick. Then, if the working shearing strength of a rivet is 10,000 pounds per square inch, the working value of one rivet in single shear is $.4418 \times 10{,}000 = 4{,}420$ pounds. If the working strength of the plate in bearing on the rivet is 20,000 pounds per square inch, the working value of the plate in bearing on one rivet is $0.3125 \times 0.75 \times 20{,}000 = 4{,}690$ pounds. The net section of the plate must have a strength as great as the smaller value found above. If the working tensile strength of the plate is 15,000 pounds per square inch, the required net width between two rivets will be $\frac{4{,}420}{.3125 \times 15{,}000} = .943$ inch. Since it is customary, in deducting rivet holes, to consider the hole $\frac{1}{8}$ inch larger in diameter than the rivet, the required distance center to center of rivets is $.943 + .75 + .125 = 1.818$, or about $1\frac{13}{16}$, inches. The working strength of the gross section of the plate for a width equal to the pitch of the rivets is $1.813 \times .3125 \times 15{,}000 = 8{,}500$ pounds. Then, since the value of the rivet is 4,420 pounds, the **efficiency of the joint** (that is, the ratio of the value of one rivet to the working strength of the gross section of the plate for a width equal to the pitch) is $\frac{4{,}420}{8{,}500} = .52$, or 52.0 per cent.

36. Double-Riveted Lap Joint.—If, other conditions being as in the preceding article, the joint is double-riveted, as in Fig. 8 (*b*), the working value of the rivets in a width equal to the pitch is, since there are two rivets, $2 \times 4{,}420 = 8{,}840$ pounds, and the required net width between rivets is $\frac{8{,}840}{.3125 \times 15{,}000} = 1.886$ inches. Then, the required distance center to center of rivets is $1.886 + .75 + .125 = 2.761$, or about $2\frac{3}{4}$, inches. The working strength of a width of plate equal to the pitch is $2.75 \times .3125 \times 15{,}000 = 12{,}890$ pounds. The efficiency of this joint is $\frac{8{,}840}{12{,}890} = .686$, or 68.6 per cent.,

which is 68.6 − 52.0 = 16.6 per cent. greater than that of the single-riveted lap joint, but this increase requires a wider plate and more rivets.

37. Triple-Riveted Lap Joint.—If, the other conditions being as in the last two articles, the joint is triple-riveted, as in Fig. 8 (*c*), the working value of the rivets in a width equal to the pitch is $3 \times 4{,}420 = 13{,}260$ pounds, and the required net width between rivets in any row is $\frac{13{,}260}{3{,}125 \times 15{,}000} = 2.829$ inches. Then, the required distance center to center of rivets in any row is $2{,}829 + .75 + .125 = 3.704$, or about $3\frac{11}{16}$, inches. The working strength of a width of plate equal to the pitch is $3.688 \times .3125 \times 15{,}000 = 17{,}290$ pounds. The efficiency of the joint is $\frac{13{,}260}{17{,}290} = .767$, or 76.7 per cent. The efficiency of this joint is, in this case, 76.7 − 68.6 = 8.1 per cent. greater than that of the double-riveted lap joint, and 76.7 − 52.0 = 24.7 per cent. greater than the single-riveted lap joint. This increase, however, requires a wider plate and more rivets.

38. Butt Joints.—In a **butt joint**, the edges of the plates are brought close together, the closer the better, and one or more cover-plates are riveted to them. Fig. 9 shows two forms of butt joints: that shown at (*a*) is a **single-welt** butt joint, and that shown at (*b*) is a **double-welt** butt joint. In each figure, *a* and *b* are the plates that are to be

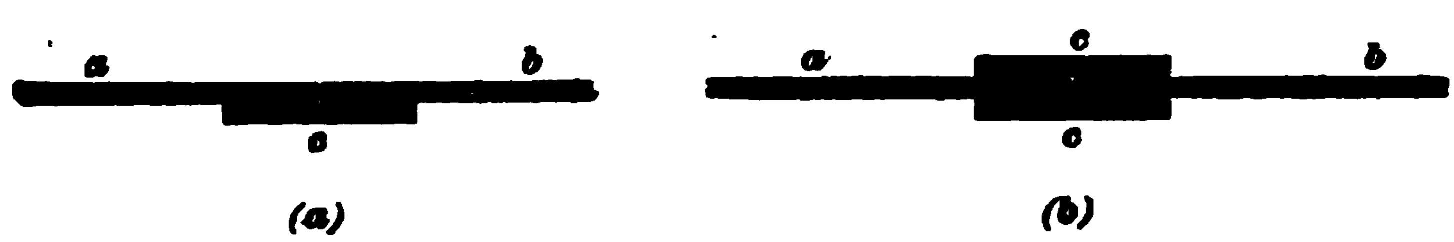
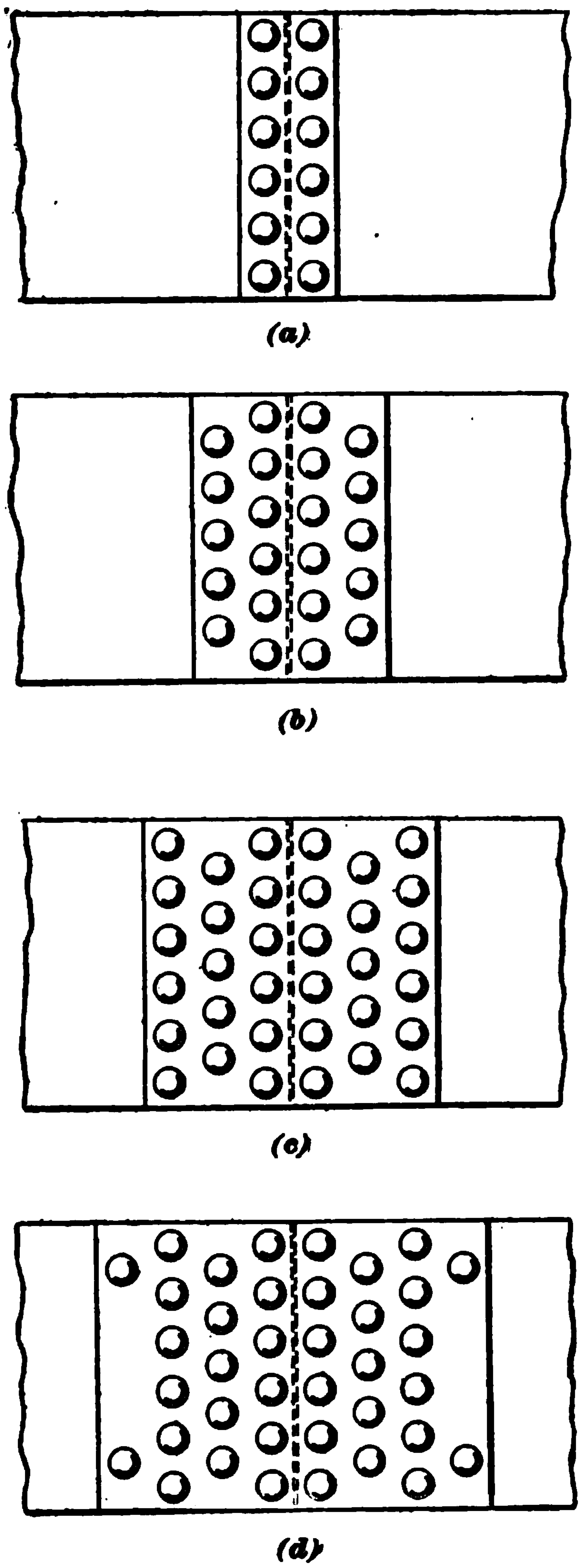


FIG. 9

joined, and *c*,*c* are the splice- or cover-plates. The joint shown at (*a*) consists simply of a lap joint on each plate, and will not be further discussed.

39. Butt joints may be **single-, double-, triple-,** or **quadruple-riveted,** as shown in Fig. 10 (*a*), (*b*), (*c*), and (*d*), respectively. The design of such joints is similar to the

design of lap joints, except that in the case of butt joints, since the rivets are in double shear, the strength of the plate in bearing on the rivet is generally the smaller, instead of the shearing strength, as in the case of the lap joint. Suppose that, in Fig. 10 (*b*), the rivets are $\frac{7}{8}$, or .875, inch in diameter (.6013 square inch in cross-section), and that the plates are $\frac{1}{2}$ inch thick. Then, if the working shearing strength of a rivet is 10,000 pounds per square inch, the shearing value of two rivets is, since they are in double shear, $2 \times 2 \times .6013 \times 10{,}000 = 24{,}050$ pounds. If the working strength of the plate in bearing on the rivet is 20,000 pounds per square inch, the working value of the plate in bearing on two rivets is $2 \times .875 \times .5 \times 20{,}000 = 17{,}500$ pounds. The required net width between rivets is $\frac{17{,}500}{.5 \times 15{,}000} = 2.333$ inches, and, since the rivets are .875 inch in diameter, the required distance center to center of rivets is equal to $2.333 + .875 + .125 = 3.333$, or about $3\frac{5}{16}$, inches. The working strength of the gross section of the plate for a



(*a*)

(*b*)

(*c*)

(*d*)

FIG. 10

width equal to the rivet pitch is $3.313 \times .5 \times 15{,}000 = 24{,}850$ pounds. Then, the efficiency of this joint is $\frac{17{,}500}{24{,}850} = .704$, or 70.4 per cent. This is somewhat greater than the efficiency of the double-riveted lap joint, but the increase is obtained by the use of more plate and about twice the number of rivets.

Butt joints are more expensive than lap joints, but, on account of their higher efficiency, they are used oftener.

40. Vertical Joints.—The stress in a vertical joint is the same as the stress in the pipe. The stress S per linear foot of the joint is given by the approximate formula

$$S = \frac{p\,d}{2},$$

where p and d have the same meaning as in Art. 28, d being in feet, and p in pounds per square foot. In designing, it is customary to find first the thickness of pipe as explained in Art. 28, and to assure a certain efficiency (usually about 60 or 70 per cent.) for the joint. Then, the vertical joint is designed for this thickness of plate, and if the efficiency comes out different from the assumed efficiency, the thickness of plate is revised and a new joint designed.

41. Horizontal Joints.—The stress in a horizontal joint is due to the weight of the standpipe above that joint and to the bending moment caused by the wind pressure. The greatest stress in pounds per linear foot of the circumference is given by the formula

$$s = \frac{60\,h_1^2 + W}{\pi\,d},$$

in which h_1 (feet) is the distance from the top of the standpipe to the joint considered; d (feet) is the inside diameter of the pipe; and W (pounds) is the weight of the standpipe above that joint. After the thicknesses of plates have been decided on, as explained in the preceding article, it is well to apply the formula to the rings near the bottom, to see that the working strength of these plates is not exceeded.

It will sometimes be found necessary to increase the thicknesses of a few rings near the bottom.

42. Choice of Joints.—The kind of joint that is used depends on the stresses and on the designer. As a rule, lap joints are used for the horizontal joints and butt joints for the vertical joints. Near the top of the pipe, the vertical joints are sometimes lapped; it is then difficult to get a tight joint.

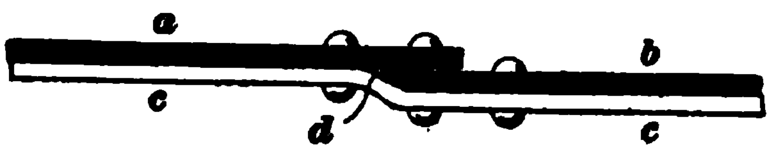
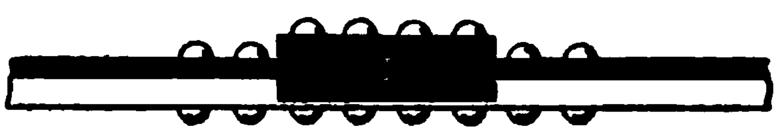


FIG. 11

This is illustrated in Fig. 11, in which a and b are cross-sections of two plates that are lap-riveted, and $c\,c$ is the upper edge of the ring below. It is necessary to hammer the plate $c\,c$ at the joint so that it will not allow water to leak out at d.


FIG. 12

This is an operation requiring great care and much work. Where a single-riveted lap joint has an efficiency large enough, it is better to use a single-welt, single-riveted butt joint, as illustrated in Fig. 12. With this joint, it is easy to prevent leaks.



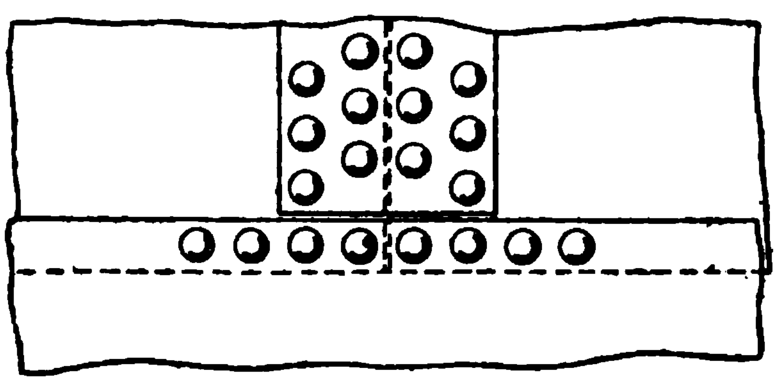


FIG. 13

The best arrangement at the junction of three plates is shown in Fig. 13. The vertical joint is a double-welt, double-riveted butt joint, and the horizontal joint is a single-riveted lap joint. The cover-plates on one side of the splice are made just long enough to fit between the rings above and below.

43. Edge Distances.—According to the best practice, the distance k (inches) from the center of a rivet to the edge of the plate should be not less than given by the formula

$$k = \frac{1}{2} + t + \frac{1}{2}d,$$

where t is the thickness of plate, and d is the diameter of the rivet, both in inches.

44. Spacing of Rivet Lines.—The distance between rivet lines for double and triple riveting is usually made three times the diameter of the rivet.

45. Bottom Plates.—The bottom of a standpipe consists of a number of plates riveted together so as to form a flat, circular bottom a little greater in diameter than the standpipe. The usual method of cutting and joining the plates is shown in Fig. 14. Since this bottom plate simply serves the purpose of preventing leaks, a very thin plate would be sufficient, but, to allow for corrosion, it is customary to use plates from $\frac{3}{8}$ to $\frac{1}{2}$ inch thick. The outer edge of the bottom is connected to the lower edge of the side by means of one or two angles in one of the manners illustrated in Fig. 15. These angles are

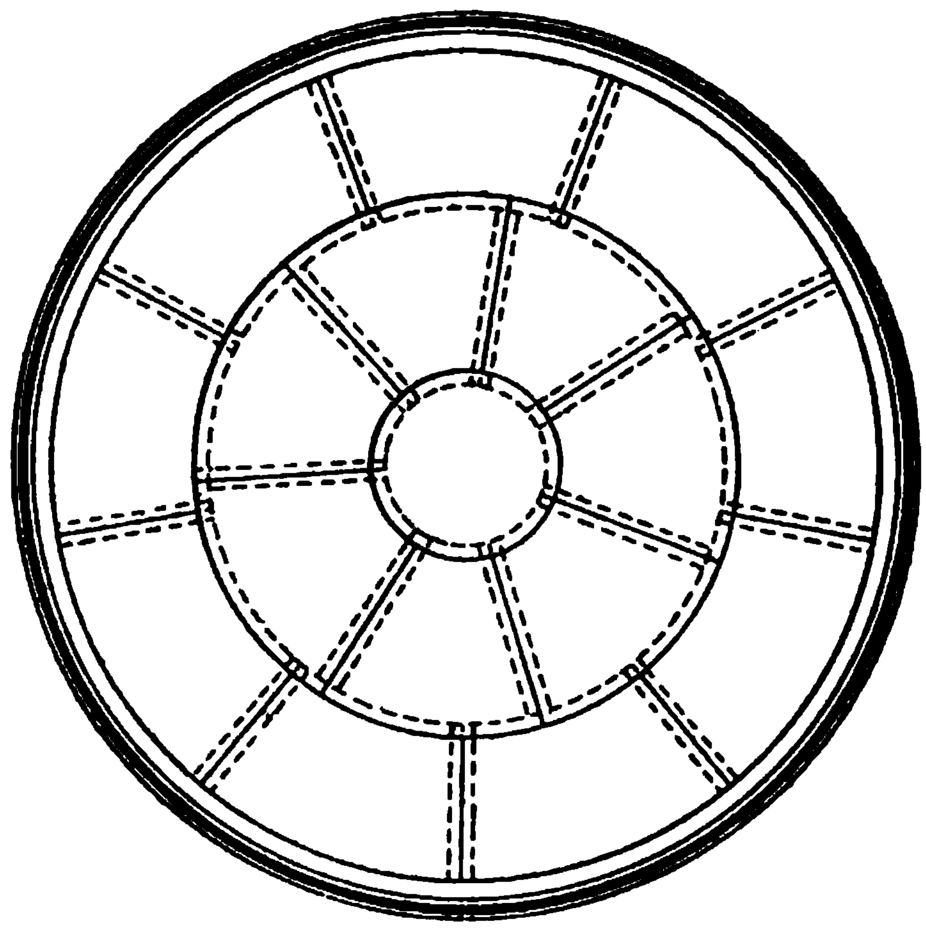


FIG. 14

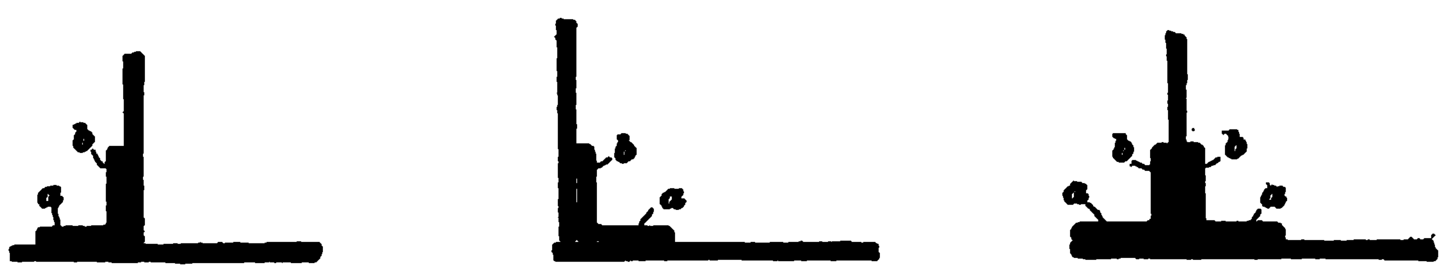


FIG. 15

bent to a curve, the horizontal leg *a* is riveted to the bottom plate, and the vertical leg *b* to the side plate.

EXAMPLES FOR PRACTICE

1. If two $\frac{3}{8}$-inch plates are joined by a single-riveted lap joint with $\frac{7}{8}$-inch rivets, what is the proper spacing of the rivets, using the same working stresses as in Art. 35? Ans. $2\frac{1}{16}$ inches

2. What is the efficiency of the joint referred to in example 1?

Ans. 51.8 per cent.

3. If two $\frac{7}{16}$-inch plates are joined by a double-riveted lap joint with $\frac{7}{8}$-inch rivets, what is the proper spacing of the rivets in each rivet line, using the same working stresses as in Art. 36?

Ans. $2\frac{13}{16}$ in.

4. If two $\frac{9}{16}$-inch plates are joined by a double-riveted double-welt butt joint with $\frac{7}{8}$-inch rivets, what is the proper spacing of the rivets in each row, using the same working stresses as in Art. 39?

Ans. $3\frac{5}{16}$ in.

5. What is the efficiency of the joint referred to in example 4?

Ans. 70.4 per cent.

6. What is the greatest stress per linear foot of the circumference at a joint 100 feet from the top of a standpipe, if the pipe is 20 feet in diameter and the portion above the joint weighs 175,000 pounds?

Ans. 12,300 lb.

ANCHORAGE AND FOUNDATION

46. Anchorage.—If the standpipe is sufficiently large in diameter and not too high, the overturning moment of the wind pressure may be resisted by the moment of the weight of the pipe. When the overturning moment of the wind pressure is the greater, it is necessary to anchor the standpipe to the masonry by means of anchor bolts. These bolts sometimes pass through the circular angle that holds the sides and bottom together, as shown at Fig. 16 (*a*); sometimes, they pass through lugs, as shown at (*b*), or brackets, as shown at (*c*), riveted to the outside of the shell. The form shown at (*a*) is used for standpipes having large diameters; a great number of bolts about $\frac{3}{4}$ or 1 inch in diameter are used. For the ordinary sizes of standpipes, the form shown at (*b*) is preferred; the number of bolts should be such that no bolt will be greater than about $2\frac{1}{4}$ inches. For standpipes whose diameters are very small in comparison with the height, say less than $\frac{1}{12}$, the form shown at (*c*) should be used. This consists of a bracket or knee brace having a web *F* about $\frac{3}{8}$ inch thick and angles at the edges about 4 in. × 3 in. × $\frac{3}{8}$ in. In this figure, *A C* is the shell and *C D* the bottom, the two being connected by the angle *B*.

The anchor bolts are inserted at H; either one or two bolts may be put in for each bracket. In the forms shown

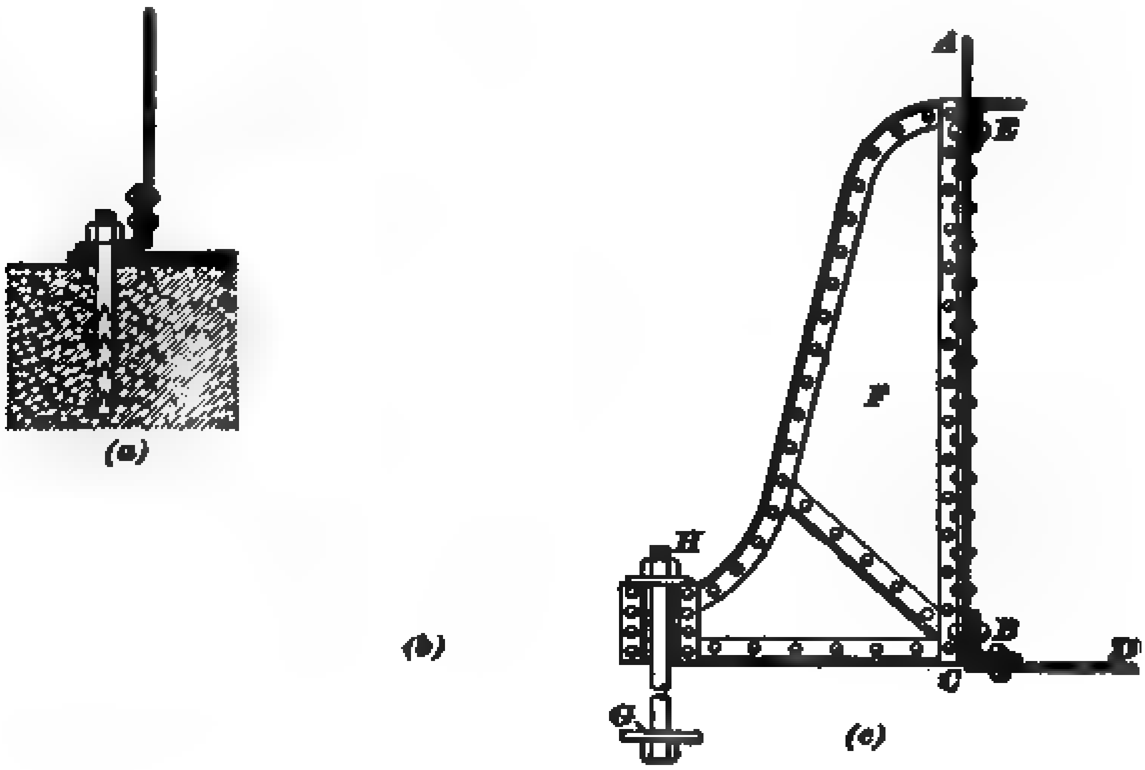


FIG. 16

at (b) and (c), the anchor bolts are continued down through the masonry, and are held in place by a plate G, 1 or 2 feet square.

47. The maximum tension S (pounds) in any anchor bolt is given approximately by the formula

$$S = \frac{60\,h^2 d - W d_1}{N d_1}, \qquad (1)$$

in which h (feet) is the height of the standpipe; d (feet) is the diameter of the standpipe; W (pounds) is the weight of the standpipe when empty; d_1 (feet) is the diameter of the anchor-bolt circle; and N is the number of anchor bolts.

When the form of anchorage shown in Fig. 16 (a) and (b) is used, the diameter of the anchor-bolt circle is seldom more than 6 or 9 inches greater than that of the standpipe, and the results will be sufficiently close if they are assumed to be equal. Then, $d = d_1$, and formula 1 becomes

$$S = \frac{60\,h^2 - W}{N} \qquad (2)$$

The required net area of the anchor bolt can be found by dividing S by the allowable tensile strength of the rod—say 15,000 pounds per square inch.

EXAMPLE.—The diameter of a standpipe is 20 feet; the height, 120 feet; the weight of the pipe when empty, 225,000 pounds; and there are sixteen anchor bolts. If the diameter of the anchor-bolt circle is 20.5 feet, what is the greatest stress in each bolt?

SOLUTION.—Since the diameter of the standpipe is 20 ft., and that of the anchor-bolt circle is 20.5 ft., the difference is so slight that they may be considered equal, and formula **2** may be used. Substituting the proper values in this formula,

$$S = \frac{60 \times 120^2 - 225,000}{16} = 39,900 \text{ lb. Ans.}$$

48. Object of Foundation.—The foundations of standpipes are designed to serve two important purposes: (1) to provide weight to prevent the structure from being blown over by the wind; (2) to provide a greater area of bearing on the soil so that the safe bearing value will not be exceeded.

To satisfy the first condition, each anchor bolt should pass through a mass of masonry whose weight is at least equal to the stress in the bolt. For instance, each of the sixteen anchor bolts referred to in the example in the preceding article should pass through not less than 39,900 pounds of masonry. Then, since there are sixteen anchor bolts, the weight of the foundation should be not less than 16 × 39,900, or about 640,000, pounds.

49. Depth of Foundation.—The foundation is usually built in the shape of a frustum of a cone, the diameter at the top being about 2 feet greater than that of the standpipe, and the diameter at the bottom depending on the kind and amount of masonry. Instead of making the foundation truly conical, it is customary to build it in a series of steps, as shown in Fig. 17. The dotted lines inside of the steps are elements to the frustum of a cone. The slope, or batter, of these lines depends on the kind of masonry used; for the ordinary kinds it is as follows:

For concrete and rubble masonry, 2 vertical to 1 horizontal.

For brickwork, sandstone, and limestone, 1 vertical to 1 horizontal.

For granite masonry, 1 vertical to 1½ horizontal.

For example, if granite masonry is used under the standpipe referred to in Art. **47** (20 feet in diameter), the top course, which will be considered 1 foot thick, will project 1½ feet on each side, and have therefore a diameter of 23 feet. If the other courses are also 1 foot thick, the diameters of the second, third, etc. will be 26 feet, 29 feet, etc., respectively. In designing, it is best to find first the amount of masonry required to provide anchorage for the anchor bolts.

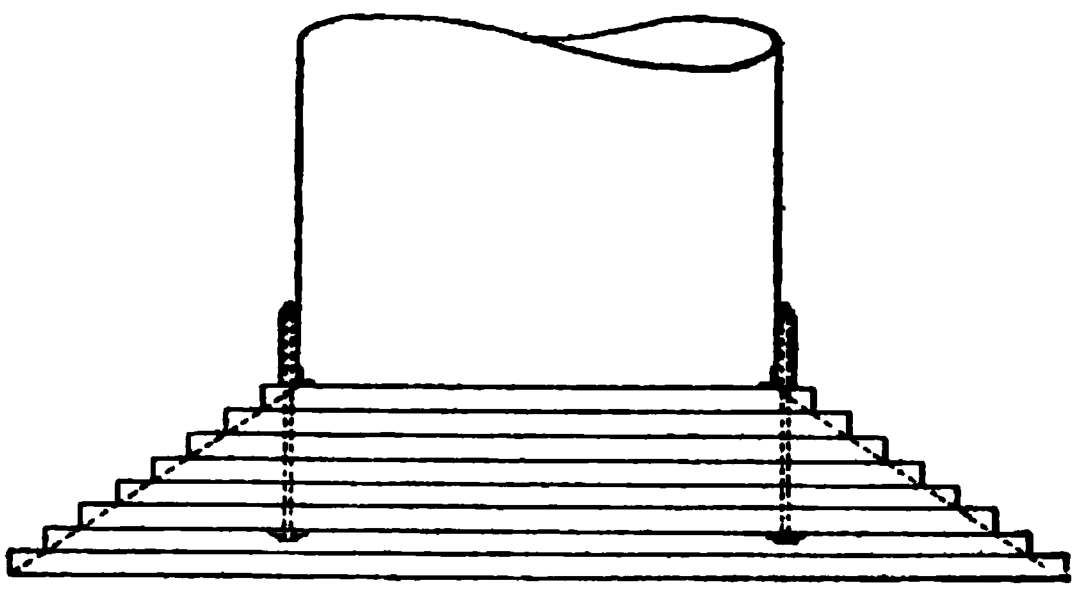


FIG. 17

If granite masonry is used, weighing 160 pounds per cubic foot, the number of cubic feet required in the above example is $\frac{640{,}000}{160} = 4{,}000$.

The depth of the foundation can now be found by adding together the volumes of successive courses, beginning at the top. The volume of each course is $\frac{\pi d^2}{4} \times 1$, or $.7854 d^2$. Since for the first course d is 23 feet, the volume is $.7854 \times 23^2 = 415$ cubic feet. Similarly:

For second course,

$.7854 \times 26^2 = 531$, total $= 946$ cu. ft.

For third course,

$.7854 \times 29^2 = 661$, total $= 1{,}607$ cu. ft.

For fourth course,

$.7854 \times 32^2 = 804$, total $= 2{,}411$ cu. ft.

For fifth course,

$.7854 \times 35^2 = 962$, total $= 3,373$ cu. ft.

For sixth course,

$.7854 \times 38^2 = 1,134$, total $= 4,507$ cu. ft.

In this case, a depth of 6 feet gives sufficient weight of masonry, so far as the anchor bolts are concerned. There is, in fact, an excess, but it is customary to go to the next nearest foot or half foot in depth. The weight of this foundation is $4,507 \times 160 = 721,100$ pounds.

50. Intensity of Bearing on the Soil.—After the depth of masonry required to provide anchorage for the anchor bolts has been found, it is necessary to compute the intensity of bearing on the soil on which the foundation rests. If this intensity exceeds the allowable working intensity of bearing on the soil, or the allowable working intensity of crushing of the masonry, as given in *Foundations*, it is necessary to increase the area of the base. This is done by increasing the depth and spreading the foundation in the same way as above. The maximum intensity of bearing s on the soil is given by the formula

$$s = \frac{Mc}{I} + \frac{W}{A},$$

in which M = overturning moment of the wind about the base of the foundation;

c = one-half the diameter of the base;

I = rectangular moment of inertia of the base about a diameter;

W = the aggregate weight of the standpipe, when full, and the foundation;

A = area of the base of the foundation.

In the example considered above, the wind pressure is $30 \times 20 \times 120 = 72,000$ pounds, and its lever arm about the base of the foundation is $\frac{120}{2} + 6 = 66$ feet. Then, $M = 72,000 \times 66 = 4,752,000$ foot-pounds. Since the diameter of the lowest course is 38 feet, c is one-half of this, or 19 feet. The moment of inertia of a circle about a diameter is $\frac{\pi d^4}{64}$; in this case, $I = \frac{\pi \times 38^4}{64} = 102,350$. The weight

of the standpipe has been assumed to be 225,000 pounds; the weight of the foundation was found to be 721,100 pounds; the weight of the water will be taken as 2,225,000 pounds; then,

$$W = 225,000 + 721,100 + 2,225,000 = 3,171,100 \text{ pounds}$$

The area of the base of the foundation is $.7854 \times 38^2 = 1,134$ square feet. Substituting the proper values in the formula gives

$$s = \frac{4,752,000 \times 19}{102,350} + \frac{3,171,100}{1,134} = 3,679 \text{ pounds per square foot}$$

This is less than the safe-bearing value of almost all kinds of dry soil, so that the foundation base need not be increased. If this pressure were greater than the allowable working pressure, it would be necessary to increase the depth 1 foot at a time, and calculate the intensity of bearing after each increase until the actual intensity of bearing came out less than the allowable working intensity.

51. Test for Stability.—After the foundation has been designed as above, it is necessary to investigate the stability, in order to see that the moment of the weight of the empty standpipe and of the foundation about the edge of the base is greater than the overturning moment of the wind. In the above example, the weight of standpipe and foundation is $225,000 + 721,100 = 946,100$ pounds, and the moment about the edge of the base is $946,100 \times 19 = 17,975,900$ foot-pounds. As this is greater than the overturning moment (4,752,000 foot-pounds), the standpipe is stable. In this case, the factor of safety against overturning is

$$\frac{17,975,900}{4,752,000} = 3.8$$

EXAMPLES FOR PRACTICE

1. The diameter of a standpipe is 18 feet; the height is 130 feet; the weight of the pipe, when empty, is 250,000 pounds; and there are twelve anchor bolts. If the centers of the anchor bolts are 2 feet outside of the shell, what is the stress in each bolt? Ans. 48,300 lb., tension

2. What is the required weight of masonry to hold down the anchor bolts in example 1? Ans. 579,600 lb.

3. If the masonry is concrete, weighing 140 pounds per cubic foot, and the diameter of the top for 1 foot is 25 feet, what is the required depth of masonry to the nearest foot, assuming that the foundation is stepped every foot? Ans. 7 ft.

4. If the weight of water in the standpipe when full is 2,300,000 pounds, what is the maximum intensity of pressure on the soil?
Ans. 5,910 lb. per sq. ft.

5. (*a*) Is the standpipe considered in the preceding questions safe against overturning? (*b*) What is its factor of safety against overturning?
Ans. $\begin{cases} (a)\ \text{Yes} \\ (b)\ 2.6 \end{cases}$

OTHER DETAILS

52. Stiffening Ribs.—Many of the failures of standpipes have come from the effect of strong winds blowing against them while empty or only partly full, and collapsing them as shown in Fig. 18. To prevent this, an angle is bent into the form of a circle of the proper size to fit inside or outside the shell, and is riveted around at the top. This angle is usually made 3 in. × 3 in. × 1½ in. for pipes up to 20 feet in diameter, and larger for larger pipes.

FIG. 18

53. Roof.—The value of a roof depends on climate and other local conditions. In warm or temperate climates, a roof has advantages in protecting the water from the rays of the sun, in reducing the amount of vegetable growths, and in preventing mischievous pollution of the water in the standpipe. In cold climates, however, a roof may be objectionable, on account of the formation of ice in the standpipe.

With low temperatures, ice forms on the surface of the water in the pipe. If that surface is low on account of a high rate of consumption (a condition likely to occur in very cold weather), ice several feet thick may form. When the temperature rises, the ice gradually melts, the water rises, and may leave some ice still attached to the sides of the pipe. With continued warm weather, the ice finally becomes free, when, on account of its buoyancy, it rises to the top with great force, and is likely to tear off the roof, if one is provided. Injury to the roof can be prevented by a careful limitation of the water level, but this reduces the capacity of the pipe.

When a roof is provided, it should be a simple construction, of steel or wood, with rafters running to a common point in the axis of the standpipe, and with short hip rafters reaching at short intervals from these to the top of the pipe. A bedplate may be fastened to the stiffening at the top of the standpipe, and sheathing boards, to be covered with tile, slate, or tin, fastened to the rafters. Galvanized roofing may be placed directly on the rafters.

54. Pipes and Valves.—The usual method of attaching the supply pipe to the standpipe is to bring the former through the masonry of the foundation, turning it upwards, by an elbow, through the bottom plate of the standpipe. The masonry is left in the form of a small tunnel, so that the pipe can be inserted or removed at any time. The elbow used is generally provided with a seat, by which the elbow is firmly carried on the masonry. The construction is illustrated in Fig. 19. There must be a water-tight joint between the bottom plate *B B* and the vertical piece of pipe, and this

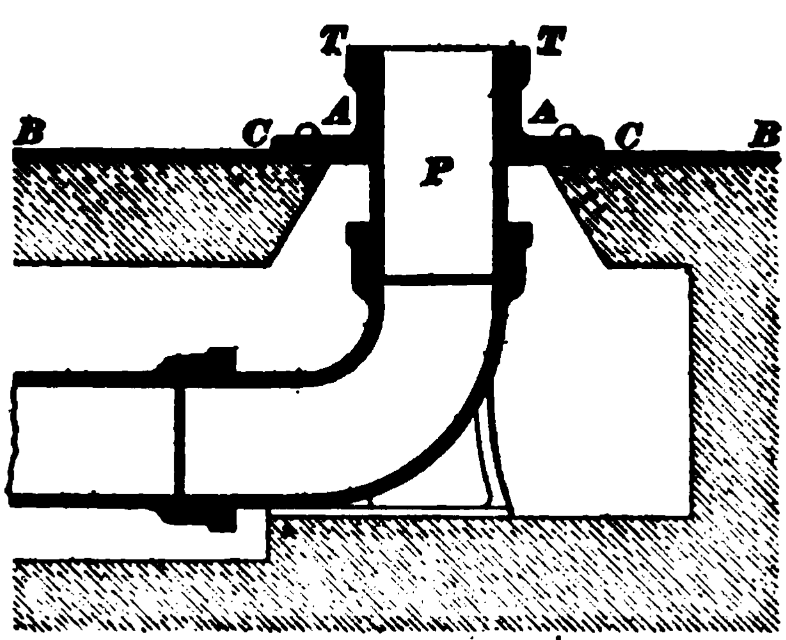


FIG. 19

is best obtained by having a special casting AA made, just large enough for the supply pipe to slip through, the special being flanged at the bottom CC and having a regular bell end at the top TT. The junction between this special and the bottom plate can then be made tight by bolting or riveting the special against a piece of packing on the floor. When the supply pipe P is put in position, projecting 8 to 10 inches above the floor, melted lead is run in between the bell of the special and the pipe. The supply pipe must be provided with a valve by which the water can be shut off or turned on. This valve is placed in a small manhole just outside of the masonry foundation.

A drainpipe is provided, by means of which water may be entirely drawn out of the standpipe. It may be made about 6 inches in diameter; it comes up through the masonry, has a flanged end, and is bolted or riveted directly against the bottom of the standpipe.

A manhole is often provided by which access may be had to the bottom of the standpipe when the latter is empty. This should be made of a heavy cast-iron or cast-steel frame well riveted to the standpipe, which may also be strengthened by a bent angle running around the opening cut in the pipe and riveted fast on the inside. The manhole cover is then screwed tight to the frame against a rubber packing.

55. Frost and Its Effects.—During severe winter weather, water freezes around the sides of the pipe, forming a hollow cylinder of ice; if warm weather follows, this ice may be detached and float in the water. If the water level rises to the top of the pipe, the ice may become fastened to the top angle and hang there even after the water falls. Afterwards, this ice may, by melting slightly, drop down and produce a sufficient concussion to damage or even ruin the structure. Another way in which ice may injure the pipe is by forming an ice tube inside the pipe and then partly thawing, especially on the south side; if cold weather follows, the water formed by the melting again freezes, and, being confined between the pipe and the ice tube, exerts a great

pressure and may split the pipe. That this does not happen more frequently is due to the fact that there is considerable elasticity to the steel, and that the ice tube is rarely thick enough to withstand the great pressure, and so gives way before any damage is done to the pipe. Several engineers, in order to prevent any possibility of damage from ice, have enclosed the standpipe inside a brick tower, with an air space of 2 or 3 feet between the pipe and the tower.

56. Painting.—In order to prevent corrosion of the metal, standpipes should be thoroughly protected by means of an impervious coating applied before rust has begun to destroy the surface. A primary coat should be applied at the shops, consisting of red lead and oil to which lampblack has been added in a small quantity. After erection, a quick-drying impervious coating should be applied. While the many patented paints in the market are probably in most cases satisfactory, an asphalt varnish or paint answers every purpose, and is cheap and easily applied. After the first coat of asphalt has thoroughly dried, a second coat should be applied. The pipe should be thoroughly repainted at intervals of not more than five years.

ELEVATED TANKS

57. When a standpipe is erected on comparatively level ground, its lower part is of no use so far as supplying water at the required pressure is concerned. For example, if the ground is level, and a standpipe 150 feet high is built, the lower 100 feet serve only to support the remaining 50 feet, and it is only the water contained in the upper 50 feet that is useful storage. In fact, there might be a bottom or partition put into the standpipe, 100 feet from the ground, only the upper part being filled, and the pipe would be quite as efficient in every particular, except stability, as when it is wholly filled. Under such circumstances, it is cheaper to build an elevated tank supported on steel columns.

HEMISPHERICAL BOTTOM

58. Form of Tank.—The first elevated tanks were built like the wooden tanks now used by railroads, with vertical sides and a flat horizontal bottom, supported by heavy beams or stringers resting on posts. The tank is now usually built in the form of a steel cylinder with a hemispherical bottom. A conical bottom has been used, but it requires more metal than a hemispherical bottom, owing to greater stresses.

The hemispherical bottom is made up, starting at the bottom, by taking a circular plate *a*, Fig. 20, dished to the proper curvature, and riveting to it plates *b*, in the shape of sectors, and also dished to the proper curvature. Successive rows of plates *c*, *d*, etc., are riveted on, one after the other, until the hemisphere is completed. The upper edge of the top row *d* is finished vertical, and is riveted directly to the inside of the vertical side plates *e*. Lap riveting is commonly used for the horizontal circular joints, and butt riveting for the joints that lie in vertical planes.

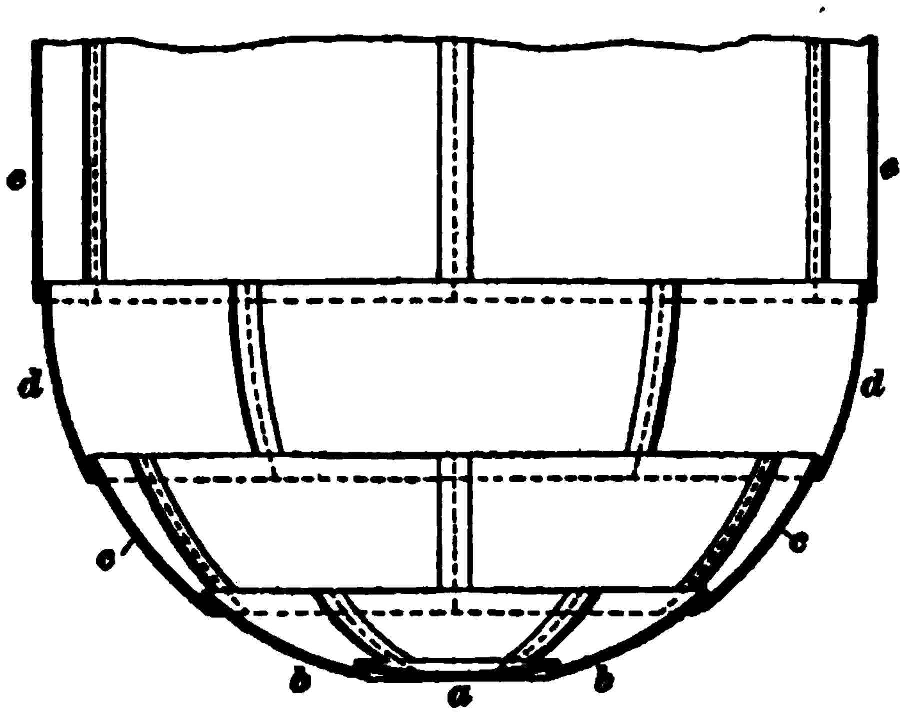


FIG. 20

The cylindrical portion of the tank is designed in the same way as for a standpipe. The difference between the design of the two lies in the bottom and in the supports.

59. Thickness of Bottom.—The required thickness t (inches) of the bottom plates in a hemispherical bottom is given by the formula

$$t = \frac{p}{4s} \times d, \qquad (1)$$

in which p = intensity of pressure at the bottom, in pounds per square inch;

s = tensile working strength of the material, also in pounds per square inch;

d = diameter, in inches, of both the tank and the hemisphere.

This is one-half the thickness that would be required for a standpipe at the same distance from the top.

For any section CD, Fig. 21, the required thickness is found by means of the formula

$$t \doteq \frac{Wd}{\pi s(d^2 - 4h_1^2)}, \qquad (2)$$

in which W (pounds) is the weight of water and bottom below the horizontal section considered, plus that of a cylinder of water having a height equal to the distance from the top of the tank to that section and a diameter equal to the diameter of that section. For example, in Fig. 21, if the thickness at CD is required, the weight is that of the metal and water below CD plus that of the water in the cylinder $CDEF$. As shown in the figure, h_1 (inches) is the depth of the section considered below the bottom of the cylindrical part of the tank. It should be observed that

$$\overline{CD}^2 = d^2 - 4h_1^2 \qquad (3)$$

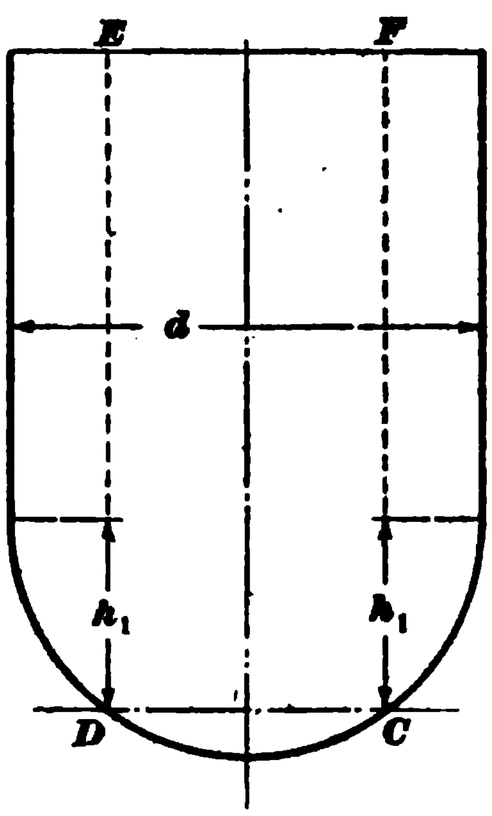


FIG. 21

It is advisable to add $\frac{1}{16}$ or $\frac{1}{8}$ inch to the required thickness as computed, in order to allow for corrosion.

EXAMPLE 1.—The diameter of a tank having a hemispherical bottom is 30 feet, and the height of the cylindrical portion is 30 feet. If the allowable working stress is 9,000 pounds per square inch, what is the required thickness of metal at the bottom of the hemisphere?

SOLUTION.—Since the diameter of the tank is 30 ft., the radius of the hemispherical bottom is 15 ft., and the distance from the top of the tank to the bottom of the hemisphere is 30 + 15 = 45 ft. Then, p, or the pressure per square inch at the bottom, is $\frac{62.5}{144} \times 45 = 19.53$ lb.

Substituting the proper values in formula **1**,

$$t = \frac{19.53}{4 \times 9{,}000} \times (30 \times 12) = .1953 \text{ in.}$$

or, adding $\frac{1}{8}$ = .125 for corrosion,

$$t = .3203 \text{ in. Ans.}$$

EXAMPLE 2.—What is the required thickness at the connection of the hemispherical bottom to the cylindrical portion of the tank referred to in the preceding example, if the weight of the curved bottom is 20,000 pounds?

SOLUTION.—At the point where the hemispherical bottom meets the vertical side, h_1, Fig. 21, is zero, and, therefore, formula **2** takes the form

$$t = \frac{W}{\pi d s} \qquad (a)$$

The weight of water contained in the cylindrical portion of the tank is $62.5 \times \frac{\pi d^2}{4} \times h = 62.5 \times 30 \times .7854 \times 30^2 = 1{,}325{,}400$ lb.

The weight of water contained in the hemispherical portion is $62.5 \times \frac{\pi d^3}{12} = 62.5 \times \frac{3.1416 \times 30^3}{12} = 441{,}800$ lb. Then, $W = 20{,}000 + 1{,}325{,}400 + 441{,}800 = 1{,}787{,}200$ lb. Substituting the proper values in equation (a) above,

$$t = \frac{1{,}787{,}200}{3.1416 \times 360 \times 9{,}000} = .176 \text{ in.}$$

or, adding $\frac{1}{8}$ = .125 in. for corrosion,

$$t = .301 \text{ in. Ans.}$$

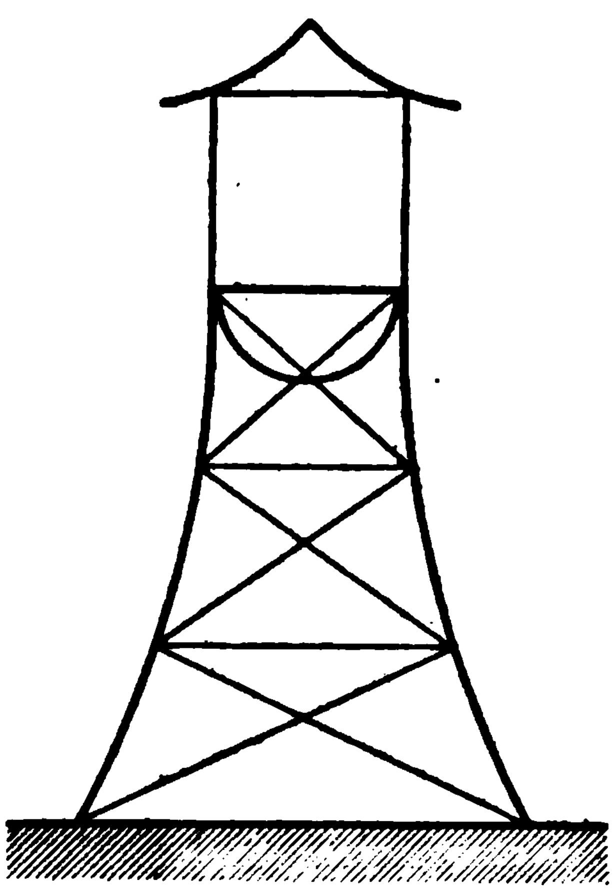


FIG. 22

SUPPORTS

60. Forms of Towers.—The customary structure for supporting a tank consists of a tower composed of four or more columns connected by horizontal struts and by diagonal lateral bracing. The posts are usually battered about 1 in 8 or 1 in 12, and are sometimes curved, or built as the chords of curves, as shown in Fig. 22. The simplest tower consists of four columns. To provide additional points of support, inclined struts are sometimes run up from each column, and connected to the

tank at intermediate points, as shown at *a a*, Fig. 23. The most common method of supporting the tank at more points, however, is by means of additional columns, six or eight being the number usually employed. The columns are spaced at equal intervals around the circumference of the tank, and are connected to it near the bottom of the cylindrical part.

61. Connection of Columns to Tank.—The two most common methods of connecting the columns to the tank are illustrated in Fig. 24. In (*a*), the column is cut off level at the top *a*, and a horizontal angle *b* is placed on the lower edge of the vertical side of the tank. The

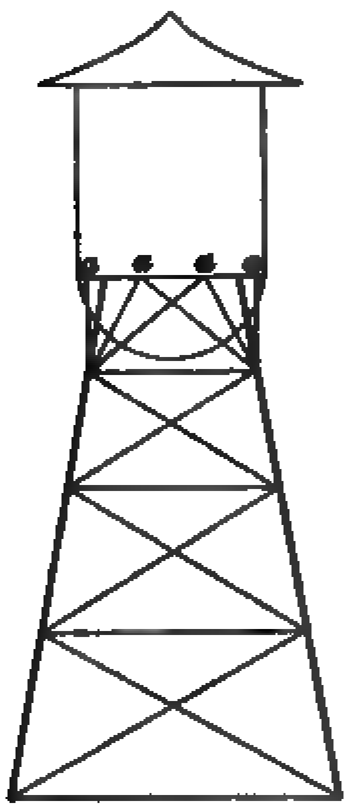


Fig. 23

Fig. 24

shell is stiffened directly over the column by means of the stiffener angle *c*. The inside of the column near the top is connected to the hemispherical bottom by means of the hitch angles *d*. In (*b*), the outside angles *e* of the column are continued straight up, and the inside of the web is cut off

vertically. The inside angles f are run vertically up the side and are riveted to the outside of the tank. The stress at each of these connections can be found in the same way as the stress in a column, as will be described presently.

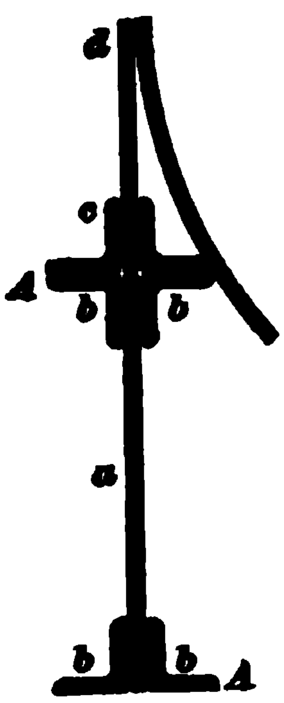


Fig. 25

62. Vertical Circular Girder.—In order to transmit the weight to the tops of the columns, the tank rests on a circular girder having a vertical web, or else the bottom ring of the plates is reinforced by circular angles riveted to them at the top and at the bottom. When the former method is used, the connection has a cross-section about as shown in Fig. 25. In this figure, $A\,A$ is the circular girder, and consists of the web a and flanges $b\,b$, $b\,b$. The bottom ring of vertical plates d forming the sides of the tank is produced below the connection of the curved bottom a short distance, and has angles c riveted to the bottom edge.

When the second method is used, the connection has the cross-section shown in Fig. 26. In this figure, the plate a is the lower ring of the vertical side, and the angles b, b are top and bottom flange angles riveted to the outside of the shell.

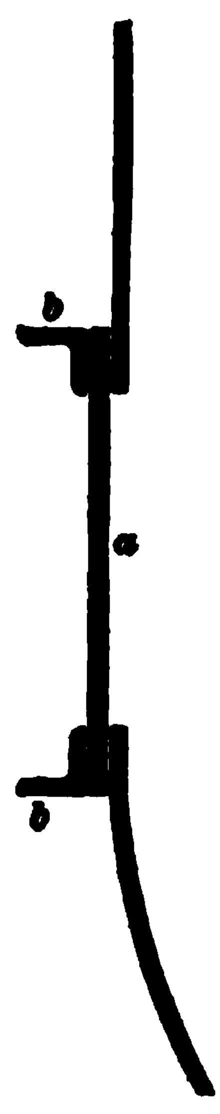


Fig. 26

63. In both of the methods just described, the web is assumed to resist the entire shear, and the flanges to resist the entire bending moment. The required net area A of each flange, in square inches, is given by the formula

$$A = \frac{M}{s\,h_g},$$

in which M is the bending moment in inch-pounds; s is the working stress; and h_g is the vertical distance between the centers of gravity of the flanges. (See *Design of Plate Girders.*)

64. The effect of the weight is to cause shearing stresses and bending moments on the circular girder. The bending

moments are positive at sections between the columns, and negative directly over the columns. The negative bending moment over the columns is about twice as large as the positive bending moments between the columns, so that the latter moments need not be considered. There is also a twisting or torsional moment on the circular girder, due to the fact that the girder is not straight between the supports; this torsional moment is much less than the vertical bending moment, and is greatest where the bending moment is almost zero. On this account, if the flanges are designed to resist the maximum bending moment, and are carried continuously around the tank, there will be sufficient section to resist the

TABLE III

REACTIONS, SHEARS, AND BENDING MOMENTS FOR VERTICAL CIRCULAR GIRDERS

Number of Points of Support	Reaction at Point of Support Pounds	Maximum Shear Pounds	Bending Moment Over Point of Support
4	$\frac{W}{4}$	$\frac{W}{8}$	$-.03415\ Wr$
6	$\frac{W}{6}$	$\frac{W}{12}$	$-.01482\ Wr$
8	$\frac{W}{8}$	$\frac{W}{16}$	$-.00827\ Wr$
12	$\frac{W}{12}$	$\frac{W}{24}$	$-.00365\ Wr$

torsional moment, so that this moment need not be considered. The maximum shears and bending moments on vertical circular girders resting on four, six, eight, and twelve columns, respectively, are given in Table III, in which W is the total weight of the tank and water, and r is the radius of the center of the circular girder.

EXAMPLE 1.—The total weight of the tank and water in example 2, Art. 59, is 1,787,200 pounds. The diameter of the tank is 30 feet.

If there are six points of support, (*a*) what is the reaction at each support? (*b*) what is the greatest shear? (*c*) what is the greatest bending moment?

SOLUTION.—(*a*) Referring to Table III, it is seen that, when there are six points of support, the reaction at each support is $\frac{W}{6}$, or, in this case, $\frac{1,787,200}{6} = 297,900$ lb., nearly. Ans.

(*b*) Referring to the table, it is seen that, when there are six points of support, the maximum shear is $\frac{W}{12}$, or, in this case, $\frac{1,787,200}{12}$ $= 148,900$ lb. Ans.

(*c*) Referring to the table, it is seen that, when there are six points of support, the bending moment at each support is $.01482\ Wr$. The radius of the circular girder is usually equal to that of the tank; so, in this case, $r = 15$ ft. Then, the bending moment is

$$.01482 \times 1,787,200 \times 15 = 397,300 \text{ ft.-lb.} \quad \text{Ans.}$$

EXAMPLE 2.—If the vertical distance between the centers of gravity of the flanges of the girder referred to in the preceding example is 4 feet, and the working strength of the net section is 15,000 pounds per square inch, what is the required net area of each flange?

SOLUTION.—The bending moment was found in the preceding example to be 397,300 ft.-lb.; that is, $397,300 \times 12 = 4,767,600$ in.-lb. The value given for h_g is 4 ft., or 48 in. Substituting in the formula,

$$A = \frac{4,767,600}{15,000 \times 48} = 6.62 \text{ sq. in., net area.} \quad \text{Ans.}$$

65. On account of the fact that the pressure of the wind on one side of the tank increases the load on the girder on the other side, the flanges should be made a little larger than called for by the formula.

66. Horizontal Circular Girder.—The columns that support the tank are usually battered so that the stresses in them have horizontal as well as vertical components, as shown in Fig. 27. In the figure, S, S are the stresses, V, V the vertical components, and H, H the horizontal components of the stresses in the columns. The vertical component is transmitted by the rivets that connect the tank to the columns. The horizontal component tends to push or dent in the shell of the tank, and it is necessary to provide a horizontal circular girder a, a, completely encircling the tank, to transmit the horizontal components of the stresses around

the tank from each column to the one diametrically opposite. This girder is usually made to act as the floor of a balcony that encircles the tank near the bottom. The railing *b b* is fastened to the outside of the girder, and a ladder *c*, shown in dotted lines, leads to the balcony.

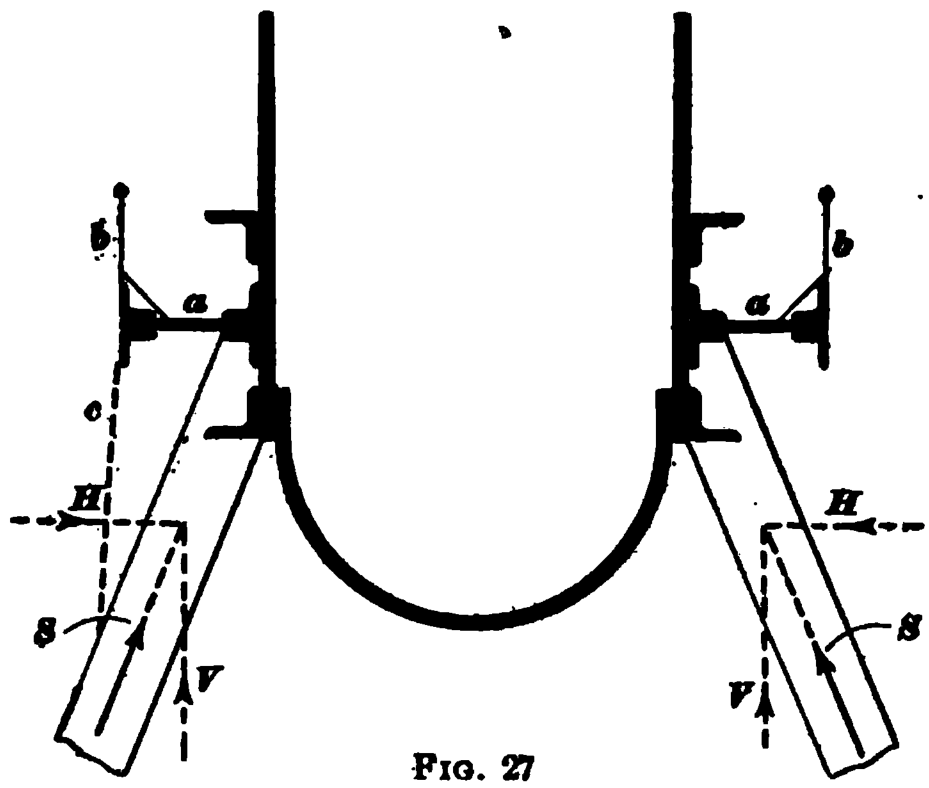


FIG. 27

67. The effect of the horizontal components of the stresses in the columns is to cause shearing stresses and bending moments on the girder, and, in addition, to cause a direct compression, which is constant throughout the circumference of the tank. The different

TABLE IV

BENDING MOMENTS, SHEARS, AND STRESSES FOR HORIZONTAL GIRDERS

Number of Posts	Bending Moment		Maximum Shear Pounds	Compression Pounds	
	At Column	Midway Between Columns		At Column	Midway Between Columns
4	$.137 H \times r$	$.071 H \times r$	$.50\ H$	$.50\ H$	$.71\ H$
6	$.089 H \times r$	$.045 H \times r$	$.50\ H$	$.87\ H$	$1.00\ H$
8	$.067 H \times r$	$.033 H \times r$	$.50\ H$	$1.21\ H$	$1.31\ H$
12	$.044 H \times r$	$.022 H \times r$	$.50\ H$	$1.87\ H$	$1.93\ H$

stresses are given in Table IV, in which H is the horizontal component of the stress in one column, and r is the radius of the neutral circle of the girder.

The bending moment is greatest at the columns, and the compression is greatest at sections midway between the

columns. The bending moment causes tension in one flange and compression in the other.

The gross area of each compression flange required to resist the bending moment is given, as before, by the formula

$$A = \frac{M}{s h_g} \qquad (1)$$

In this case, h_g is measured horizontally on a radial line.

The direct compression in the girder increases the compression in the compression flange and decreases the tension in the tension flange. It is assumed that it is equally distributed between the two flanges, so that the additional area A_1 of each flange required to resist this compression C is given by the formula

$$A_1 = \frac{C}{2s} \qquad (2)$$

Then, $A + A_1$ is the required area of each flange. It is necessary to find the required area at the columns and at sections midway between the columns; the area of the flange is then made equal to the greater value thus found.

EXAMPLE 1.—Let the horizontal component of the stress in each column of a tank 30 feet in diameter, supported on eight columns, be 37,500 pounds, and let the radius of the neutral circle of the horizontal girder be 16 feet 6 inches. (*a*) What is the maximum bending moment on the girder? (*b*) What is the maximum compression?

SOLUTION.—(*a*) Referring to Table IV, it is seen that the bending moment is greatest at the column, and is equal to .067 Hr. In the present case, since H = 37,500 lb. and r = 16.5 ft., the bending moment is

$$.067 \times 37{,}500 \times 16.5 = 41{,}500 \text{ ft.-lb.} \quad \text{Ans.}$$

(*b*) From Table IV the compression is found to be greatest midway between the columns, and equal to 1.31 H. In the present case, since H = 37,500 lb., the compression is 1.31 × 37,500 = 49,100 lb. Ans.

EXAMPLE 2.—Suppose that the bending moment and the compression found in the preceding example are obtained at the same section. Then, if the allowable intensity of stress were 15,000 pounds per square inch, and the distance between the centers of gravity of the flanges were 33 inches, what would have been the required area of each flange at that section?

SOLUTION.—The area required to resist the bending moment is given by the formula

$$A = \frac{M}{2h_g}$$

It was found in the preceding example that $M = 41,500$ ft.-lb. $= 41,500 \times 12 = 498,000$ in.-lb. Then, since $s = 15,000$ lb. per sq. in. and $h_g = 33$ in.,

$$A = \frac{498,000}{15,000 \times 33} = 1.01 \text{ sq. in.}$$

The area required to resist the direct compression is given by the formula $A_1 = \frac{C}{2s}$. In the preceding example, it was found that $C = 49,100$ lb.; then,

$$A_1 = \frac{49,100}{30,000} = 1.64 \text{ sq. in.}$$

The total required area is, then,

$$1.01 + 1.64 = 2.65 \text{ sq. in. Ans.}$$

68. Stresses in the Columns.—The stress in each column is composed of two parts; namely, that due to the weight of the water and tank, and that due to the wind pressure. The vertical component of the stress due to the weight can be found by dividing the total weight by the number of columns. The wind pressure decreases the compression on the columns on the side from which the wind is blowing, and increases the compression in the columns on the side toward which the wind is blowing. The vertical components of the wind stresses in the columns are usually found by considering a horizontal plane cutting all the columns, and applying the equation $\Sigma M = 0$ to all the wind forces acting on the tank and portion of the tower above that plane, taking for the axis of moments a line at right angles to the direction of the wind and passing through the center of the section. Let Fig. 28 represent a horizontal section at any point in a tower, and let A, B, C, D, etc., represent the columns. The wind stresses in the columns are greatest

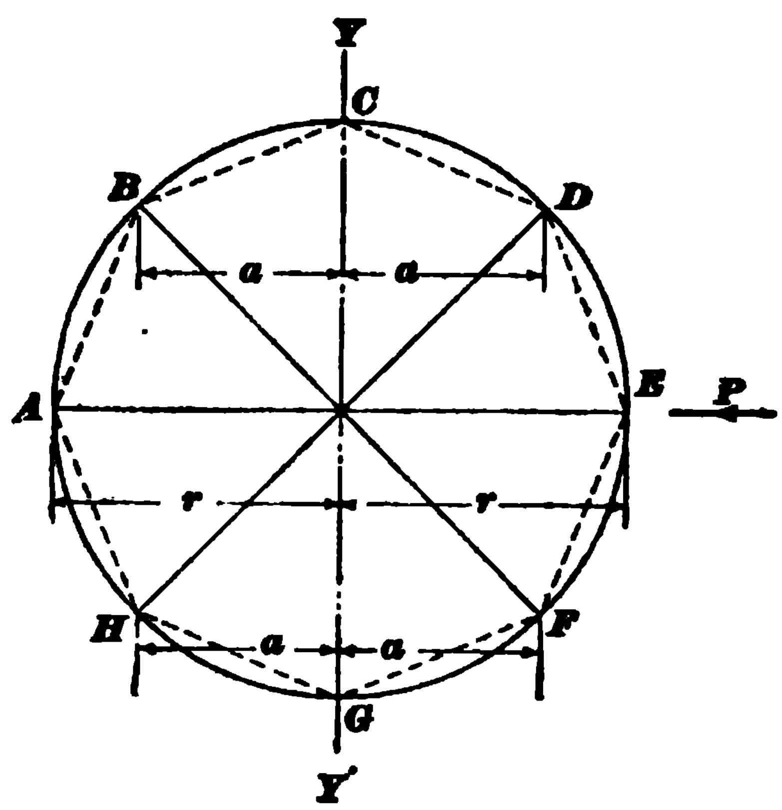


FIG. 28

when the wind is blowing in a direction parallel to a diagonal of the polygon of which the columns are the vertexes. The axis of moments in the figure is YY_1, passing through the center of the section and at right angles to the direction of the wind. The moments of the horizontal components of the stresses in the columns about the axis of moments will be zero, so that they need not now be further considered. The vertical components of the stresses in the columns vary directly as their distances from the axis of moments. If M is the moment of the wind pressure about the axis of moments, r the radius of the circle passing through the centers of the columns at the section considered, and a the distance of any column from the axis of moments, the greatest value of the vertical component F_v' is given by the formula

$$F_v' = \frac{Mr}{\Sigma a^2} \qquad (1)$$

in which Σa^2 denotes the sum of the squares of the distances of all the columns from the axis of moments.

If W is the weight of the tank, water, and portion of the tower above the section considered, and N is the number of columns, the vertical component F_v of the greatest compression in any column is given by the formula

$$F_v = \frac{W}{N} + \frac{Mr}{\Sigma a^2} \qquad (2)$$

This formula should be applied to the tower at the top, and at the connection of each set of transverse struts. In addition, it should be applied at the tops of the piers on which the columns rest, in order to get the greatest load on any pier.

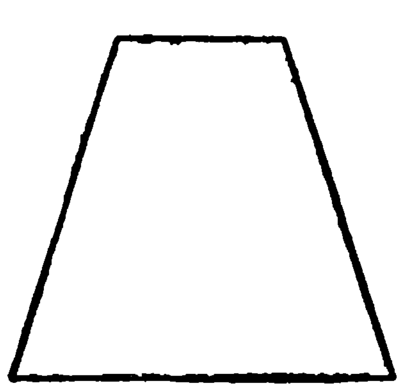


FIG. 29

69. Foundations.—The foundation for a tank usually consists of a number of piers having a cross-section about as shown in Fig. 29. The opposite sides are usually battered the same amount, but they have been built, in some cases, with different side batters, so that the center line of the column produced will intersect the center of the base, as shown in Fig. 30. The required area of bearing is

found by dividing the greatest load on the pier by the allowable intensity of bearing on the soil.

70. Anchorage.—It is also necessary to provide sufficient anchorage so that the tank will not blow down when it is empty. If W_1 represents the weight of the tank and tower (the weight of water is not considered), the negative reaction U, or uplift at the foot of any column, is given by the formula

$$U = \frac{W_1}{N} - \frac{Mr}{\Sigma a^2}$$

To allow for the effect of sudden gusts of wind, it is customary to make the weight of each pier at least equal to $2U$, thereby insuring a factor of safety of at least 2.

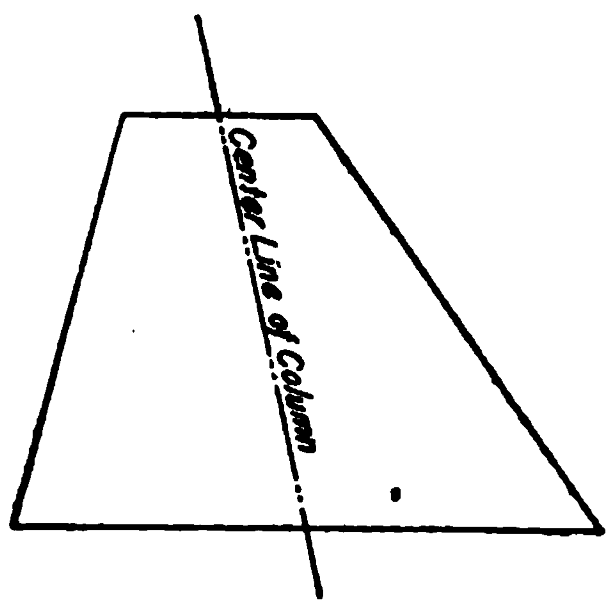


FIG. 30

The base of each column is bolted to the pier by anchor bolts large enough to transmit the reaction, and passing almost completely through the masonry.

71. Wind Pressure.—The wind pressure on the cylindrical portion of the tank is computed in the same way as for standpipes. The pressure on the roof and on the hemispherical bottom will be sufficiently allowed for, if it is assumed to be the same as that on a height equal to one-half the diameter. Then, the total area of the tank exposed to the wind is $d\left(h + \frac{d}{2}\right)$. As in the case of a standpipe, the center of the wind pressure may be taken halfway up the cylindrical portion.

There is also to be considered the wind pressure on the columns and bracing. This will be sufficiently allowed for if the area exposed to the wind is taken equal to the sum of twice the widths of all the columns multiplied by the height of the tower under consideration. This makes ample provision for the exposed area of the bracing.

72. Stresses in Wind Bracing.—It is impossible to calculate exactly the stresses in the members of the wind bracing. This is due to the fact that there is a complete

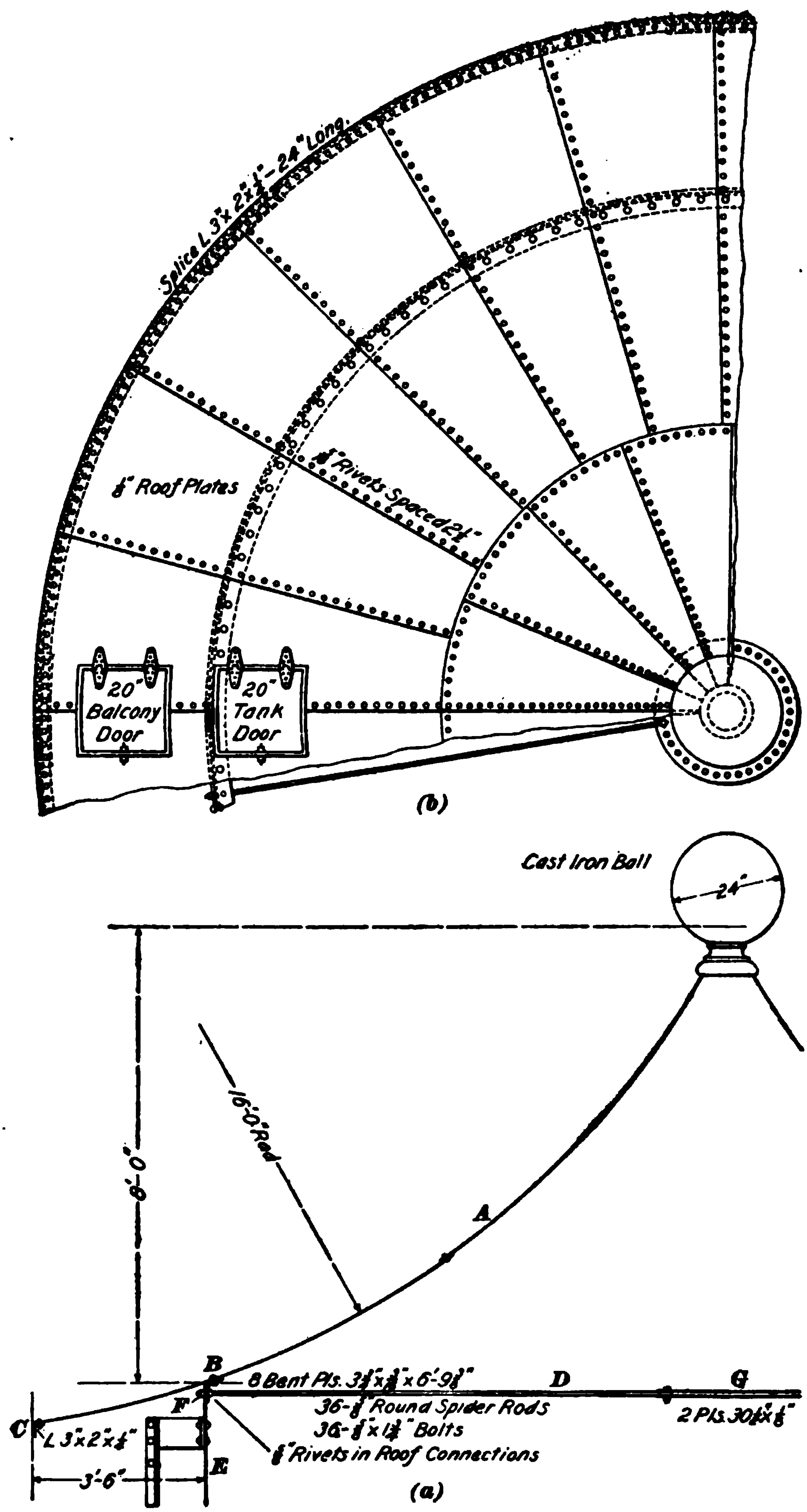

Splice L 3"x 2"x
-24" Long.
Roof Plates
Rivets Spaced
20" Balcony Door
20" Tank Door
(b)
Cast Iron Ball
24"
16'-0" Rad
8'-0"
A
B
8 Bent Pls.
D
G
C
F
E
L 3"x 2"x
36-
Round Spider Rods
Bolts
Rivets in Roof Connections
2 Pls.
3'-6"
(a)

FIG. 31

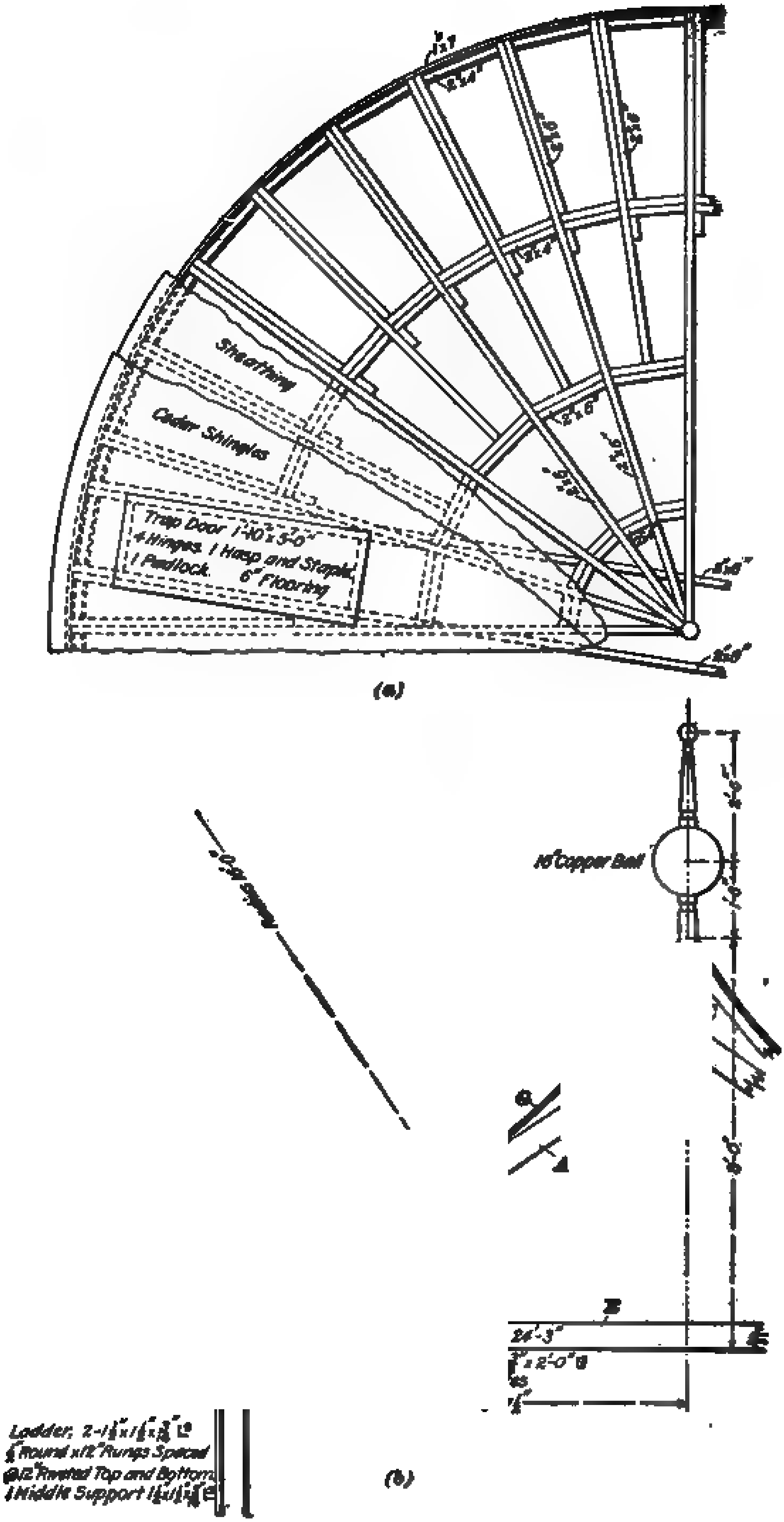
system of diagonals and horizontal struts in each of the planes formed by two consecutive columns. The manner in which the wind pressure is divided among these systems depends to a great extent on the workmanship and fit of the various members, so that it is practically useless for the designer to waste time attempting to calculate the stresses accurately.

The best that can be done is to make an assumption that will surely be on the safe side. This will make the members of the bracing a little heavier than absolutely necessary; but since they form a very small part of the whole structure, the increase in cost will be comparatively small. It is safe to assume that the compressive stress in each horizontal strut, and the horizontal component of the stress in each diagonal, is equal to one-half the total wind pressure on the entire structure above the member under consideration.

DESIGN OF ROOF

73. Figs. 31 and 32 show complete designs for roofs of small standpipes or tanks. The same types can be used for larger tanks. Fig. 31 shows a metal roof made up of a number of pieces of $\frac{1}{8}$-inch roof plates riveted together with $\frac{5}{16}$-inch rivets and fastened at the top edge of the tank at eight points of the circumference by bent plates *B*. A small angle *C* is riveted to the under side of the roof plates at the bottom outside edge, to make a finish and to increase the stiffness. To take up the outward thrust of the roof, thirty-six light rods *D* are spaced around the circumference, the outer end of each passing through the plates *E* of the tank and held in place by nuts *F*, and the inner end being looped and connected to two circular plates *G* by $\frac{5}{8}$-inch bolts. Two trap doors, shown in Fig. 31 (*b*), are provided, one opening from the balcony outside the tank and one from the inside of the tank itself.

Fig. 32 shows a wooden and shingle roof, made by fitting $2'' \times 6''$ rafters *A* from the top of the tank *B* to a center pole *C*, the lower ends resting on and bolted to a $5'' \times 3''$



Sheathing
Cedar Shingles
Trap Door
4 Hinges, 1 Hasp and Staple
1 Padlock.
Flooring
(a)
Copper Ball
(b)

FIG. 22

angle D. Horizontal $2'' \times 6''$ pieces E are used to brace the top of the tank and to support the lower part of the roof. The curvature is formed on smaller auxiliary rafters F sawed to circles and spiked on the main rafters. The sheathing G is 1 inch thick, laid on top of the rafters, and the shingles are laid on top of them.

EXAMPLES FOR PRACTICE

1. The diameter of a tank having a hemispherical bottom is 35 feet, and the height of the cylindrical part is 40 feet. If the working stress is 9,000 pounds per square inch, what is the required thickness of metal at the bottom of the curved bottom? Ans. .291 in.

2. If the total weight of the tank and water in the preceding question is 3,250,000 pounds, and there are eight points of support, what is the reaction at each support, if only the weight is considered?
Ans. 406,250 lb.

3. What is the greatest bending moment on the vertical circular girder that supports the tank referred to in the two preceding questions, if the diameter of the girder is equal to that of the tank?
Ans. 470,400 ft.-lb.

4. If the horizontal component of the stress in each column of the tank referred to in the preceding questions is 50,000 pounds, and the radius of the neutral circle of the horizontal circular girder is 19.5 feet, what is the greatest bending moment in the girder?
Ans. 65,300 ft.-lb.

SEWERAGE
(PART 1)

PRELIMINARIES

1. Divisions of the Subject.—For convenience, the subject of sewerage will be divided into two parts: the first part will deal with the general principles and theories on which the design and construction of sewers depend; the second part will treat of those methods of construction that have been approved in practice.

2. What Must be Removed by Sewers.—A good sewerage system should provide for the prompt removal of all water from the surface and subsoil—whether rainfall or ground water—all garbage, street sweepings, solid kitchen and factory waste, decaying vegetable matter, and all excremental and liquid refuse. By such means, putrefying matter and stagnant water, which not only generate disease germs, but also make their continued existence possible, are effectually removed. The soil is thus rendered dry and wholesome, and the air is purified.

3. Sewerage Systems.—From the engineering standpoint, the subjects of drainage and sewerage are so intimately associated as to be almost inseparable. In general, however, it may be stated that, while the subject of sewerage refers principally to the removal of excremental or human refuse and other waste matter common to human habitations, the subject of drainage properly relates to the removal of storm water from the surface and subsoil. All water given

by rainstorms may, without impropriety, be called **storm water.** This name, however, is commonly applied to the water from rainstorms that does not soak immediately into the ground nor evaporate, but flows away over the surface through natural channels or artificial conduits.

Sewers may be designed to carry storm water alone, house refuse alone, or both. Those that carry storm water alone are called **storm sewers;** those that carry human refuse alone are called **house sewers.** The system involving storm sewers is called the **storm-water system,** and that involving house sewers is called the **separate system,** to indicate that the house sewage is separated from the rainwater. Cities often use a **combined system,** through the pipes of which both storm water and house refuse are removed.

4. A storm-water system should be adequate for the prompt removal of the rainfall from the surface during violent storms, including also such animal and vegetable refuse from the streets as will necessarily be removed with the storm water. If this is accomplished, and the drains are located at sufficient depth, efficient drainage will be provided for the subsoil.

The separate system should be able to carry off promptly from houses all sink, laundry, and closet wastes, without offensive odors, and without interruption. It should keep itself clean, that is, free from deposits; it should not pollute the soil through which the pipes pass; and it should have an outlet that is without objection.

5. Comparison of the Separate and the Combined System.—For some conditions, the separate system of sewerage possesses certain material advantages over the combined system. The advantage of the former system that most strongly appeals to the taxpayer is its reduced cost, which is generally from one-eighth to one-half the cost of the combined system for corresponding conditions. This statement relates to the system of sewage conduits. It is evident, however, that where the sewage must be pumped,

or where it is to be utilized on a sewage farm, or by any process purified, the expense of the process will be very materially reduced by the exclusion of the storm water. The separate system, if properly constructed, will more thoroughly meet the requirements for the efficient removal of house sewage, and be more strictly sanitary, than the combined system. In the former system, the sewers, being of small section, will carry a comparatively constant volume each day, in dry as well as in wet weather, and this tends to prevent permanent deposits; the sewers will have more uniform velocities of flow; and for this reason, and also on account of their freedom from street detritus, such as gravel and sand, they can generally be constructed with flatter grades than would be required in the combined system. Moreover, when flushing is necessary, the same degree of cleanliness can be attained with less water in the separate than in the combined system.

6. For the purpose of illustrating the relative capacity that must be provided where storm water is admitted, as compared with the capacity required where it is excluded, and also for the purpose of comparing the work accomplished in each case, a rough computation will here be made. Suppose that, with a population of 10,000 on an area of 1 square mile, each person contributes an amount of sewage equal to 75 gallons a day of 16 hours. It will be assumed that, if storm water is to be removed, the capacity of the sewers must be such as will discharge 1 inch in depth of rainfall per hour from the entire area; this is a fair allowance for this density of population under ordinary conditions. It will also be assumed that about 15 inches in depth of rain over this area will find its way to the sewers during 1 year.

Computing the rates of discharge, it is found that sewage proper will be discharged at the rate of 9.75 cubic feet per acre per hour, and that storm water will be discharged at the rate of 3,600 cubic feet per acre. The discharges, and consequently the relative capacities required, will be in the ratio of 1 to 370.

An analysis of these results shows that the separate system, with $\frac{1}{870}$ of the capacity of the combined system, removes the foulest portion of the sewage. The removal of sewage proper is a daily convenience, while the inconvenience resulting from flooded gutters is suffered only at comparatively remote intervals, and the conformation of the ground is often such that storm water is not naturally concentrated, but finds its way to many natural outlets, a comparatively small volume being discharged at any one point.

7. It is to the considerations just stated that the popularity of the separate system and its adoption in many of the smaller cities are due. This system is also well adapted for towns and villages that are built on porous soil, which allows the storm water to be readily discharged at convenient outlets. It is, moreover, preferable where the sewage has to be pumped or purified. Whether it should be adopted for cities characteristically opposite to the above in soil, topography, gradients, and facilities for sewage disposal without treatment, is a matter that depends on local conditions.

SURVEYS, MAPS, AND GRADES

SURVEYS

8. Preliminary Investigations.—Before undertaking to formulate a plan for any system of sewers, the entire territory under consideration, and possibly adjoining areas that may later form a part of the same system, should be carefully studied. Levels should be taken over it; the character of buildings and manufactures should be noted; the nature of the surface, the amount of impervious covering, the texture of the subsoil, and the depth of ground water should be investigated. The present and prospective water supply and population should be learned as nearly as possible. The rate of local rainfall is a most important factor in the size of storm sewers. The velocity and volume of flow of streams

into which the sewage may be discharged must be investigated, as these hydraulic factors often determine the place of discharge. After all the facts that have a bearing on the question of sewerage have been gathered, it may be decided whether the separate or the combined system, or modifications of one of these systems, will be best adapted to the conditions.

9. Surveys and Levels.—Usually, surveys and maps of the territory have already been made for other purposes, and are available for use in the preliminary sewer investigations, or at least sewer maps may be prepared from them without entirely new and independent surveys. This is generally not true of levels. It is usually necessary to run a new and independent line of levels over the whole area; and it is also generally necessary to make supplementary surveys for outlets and for main sewers along depressions not coinciding with the streets. In making preliminary studies, the barometer method, explained in *Water Supply*, Part 3, can be used to great advantage. By this method, with the use of a horse for getting about, it is possible to obtain levels throughout a large city in 2 or 3 days—levels that will afterwards check, even at places where the maximum discrepancy is found, to within a few feet. It is desirable to make these preliminary surveys and keep the records of them in such a way that they will be of the most service in actual construction. It is also very desirable to use the same bench marks in the final construction as are used in taking the preliminary levels.

10. Datum Plane and Controlling Bench Mark. Before the levels are taken, a datum plane should be assumed, to which all elevations should be referred. The datum should be lower than the lowest point in the proposed sewers, and some controlling bench mark should be assumed, as nearly in the center of the area under consideration as possible. Generally, it is convenient to make this bench mark some substantial point on a public building. If a datum plane has been established for other surveys in the city, the same datum should be used for the sewer work. This will

make it possible to utilize, without recomputing elevations, much of the information that has been gained in other surveys, and the levels that are taken for the sewer work can also be conveniently utilized for other purposes.

11. Auxiliary Bench Marks.—After the datum plane and the controlling bench mark have been chosen, a system of bench marks should be established at short intervals over the territory. This work should be done carefully, and it is better to do it separately than to take surface levels at the same time. When establishing these bench marks, complete return circuits should be made, and the lines should be run in both directions, taking all bench marks and turning points in each direction; the elevations that are thus computed for each bench mark, if they are reasonably close, may be averaged, and a table of established elevations of bench marks then made out for use in all the preliminary levels, as well as in the final location and construction of the sewers. For details regarding the correction of levels, etc., see *City Surveying.*

12. Surface Levels.—After having determined the elevation of the bench marks, the surface levels should be taken, starting from one of these established bench marks, noting the reading on all the bench marks that are passed in the circuit, and finally closing on an established bench mark. In all cases where it is possible to do so, the field notes should show a closing on some well-established point, and the error in closing should be recorded in the notes. These closing measurements are often of the greatest use in locating errors. Whatever error is noted in closing on an established and checked bench mark should be dropped, and the work taken up anew from the bench mark as a basis.

When taking the surface levels, it is well to note the elevations at each side of the street. These are really what determine the elevation of the sewer, the elevation of the center of the street being of importance only in determining the depth of the cut after the proper elevation of the sewer has been fixed. The elevation of the basement floors that

have already been built, and, if it is probable that the territory will develop, the depth of basement that may be expected in future buildings should be noted, and also the distance back from the street line to basements or buildings. The elevation of basements will best be determined by taking a reading with the leveling rod on the main-floor line of the building as the level party is passing along the street. After this, at some convenient time, measurements can be made from the main-floor line to the bottom of the basement; by following this plan, the progress of the level party will not be delayed.

The preliminary surface surveys are not usually those on which the final profiles and estimates are based. It is not generally practicable at this stage of the work to determine the precise location of the sewers in the streets; and they may be finally located at one side of the line on which the levels are run. The final surface profiles are made after a line has been adopted and actually staked out on the ground.

13. Special Surveys and Investigations.—The topography of the city is often such that some of the main lines of sewers cannot follow the streets entirely, but must be run along valleys that form the natural drainage channels for the surface water and are also the points at which the sewage proper can best be concentrated. This is the case where deep valleys intersect the streets in an irregular manner, so that sewers cannot well be brought across them, but must descend into them abruptly from either side, and must be, at these points, gathered into a main outlet following the natural depression. Some special surveys and preliminary location of the line are generally necessary in such cases. Sometimes, there are bridge abutments and similar obstructions that must be investigated, in order that the most feasible route for the main sewer may be determined.

The cost of sewers is materially greater where they are built in unstable ground and in wet trenches, and, therefore, the character of the soil and subsoil should be thoroughly investigated.

The location of outlets often becomes apparent on a superficial examination, and generally is determined before the preliminary work has progressed far. If the outlets are to be in streams, the condition of the channel, the currents at all stages of the water, and the difficulty that may be experienced in freshets or when ice is running should be noted.

There are generally water mains, gas mains, and conduits of various kinds, which should be avoided as far as possible in locating the sewers. They often lie in such positions that they must be considered in determining the grade of sewers crossing them. Their positions and elevations should be carefully determined and recorded.

MAPS

14. Construction of Map.—With the surveys and levels at hand, and with such maps as may be available for a basis, a topographic map of the city is made. This map should be on a scale of not more than 400 feet to 1 inch; for a small city, where the map would not be too large, a larger scale may be used to advantage. The State Board of Health of New York suggests a scale of 250 feet to 1 inch as desirable. For undulating ground, the contours may be drawn every 5 feet; for flat ground, every foot. A convenient way to proceed is first to make a map of the area on tracing cloth, showing the elevations, the contours, and everything else that is to be shown in the final plan, except the sewers themselves. Blueprints can be made from this tracing, and on them the plans may be developed roughly in pencil. In developing the plans, it is convenient to make figures directly on the map; as the work progresses, there will be many changes and corrections, and perhaps several alternative plans carried nearly to completion, and after further investigation, abandoned. When the final arrangement is chosen, the sewers may be added to the tracing, making the map complete.

15. District on High Ground.—With the topographic map finished, the physical character of the district should be

carefully studied. If the location of the district is near a summit or at a high elevation, with the surface sloping in such a manner as to afford ample channels of natural drainage, the problem of sewer design will be very simple. In that case, the lines of sewers are located along such streets as most nearly follow the natural drainage channels, leading to such natural outfall as may be available. It is often found that the positions, directions, and grades of the streets are to some extent determined by the natural channels of drainage.

16. District Without Natural Outlet.—If the district is situated in a valley not having a good natural drainage

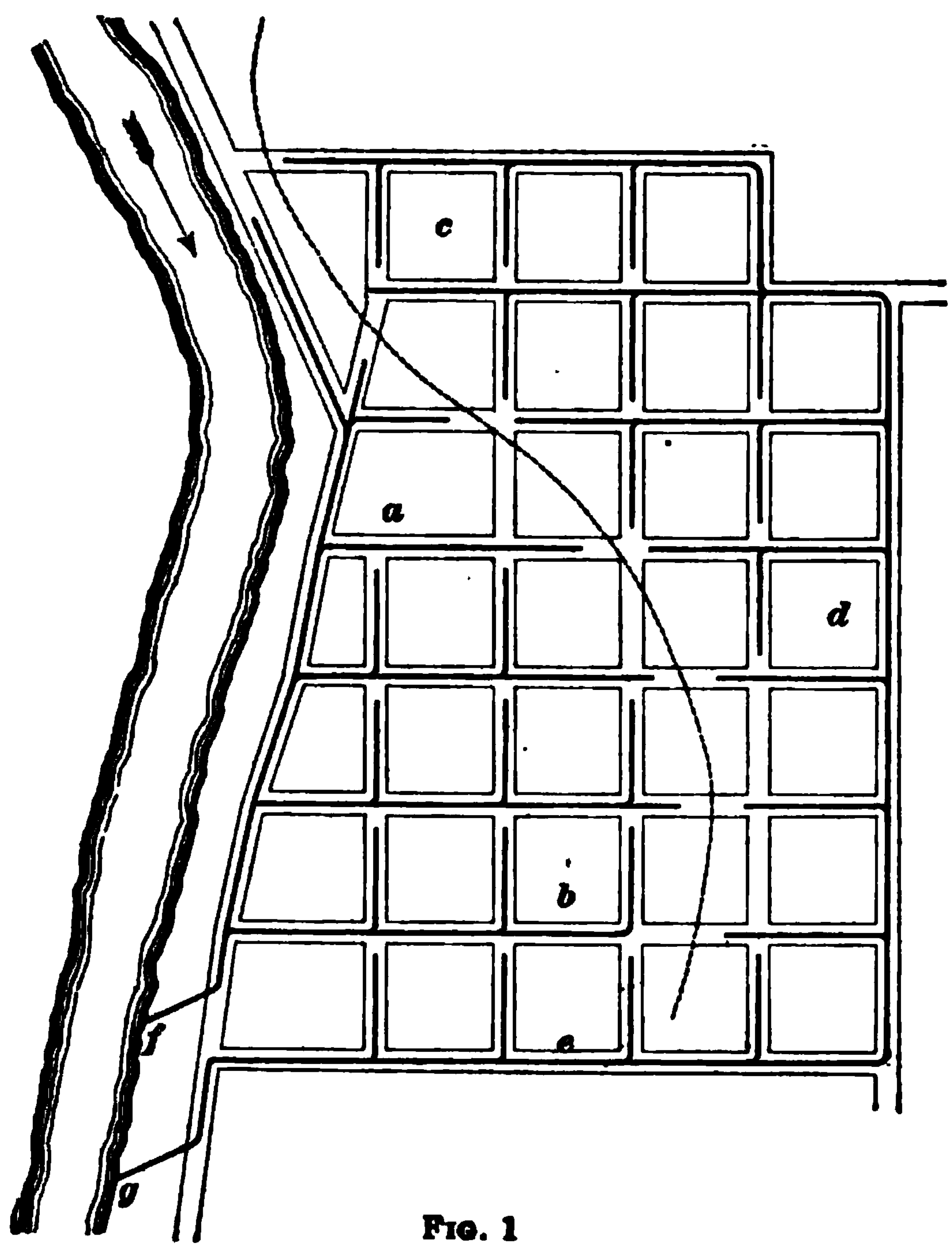


FIG. 1

outlet, the problem may be much more complicated, and the expense of the construction of a sewer system greatly increased. If the sewage is to be discharged into a running

stream passing through the town, it should be delivered to the stream at a point *below* the town, whatever may be the direction of the natural drainage. This may require inconvenient and circuitous courses for the sewers, as well as deep excavations. A case of this nature is represented in Fig. 1. As there shown, the town is situated on the bank of a stream. A crest of elevated ground, in the position indicated by the dotted line, separates the town into practically two drainage districts. The outfall for the district *a b* is at *f*, while the sewage from *c* passes around by the circuitous course *d e* and is discharged at *g*. The sewage from *c* could not pass by a direct course to the outlet without an exceedingly deep excavation through the high ground.

It is always necessary to study the map until the elevations and depressions and the ridges and valleys are thoroughly impressed on the mind. It is sometimes of help to draw, on one of the blueprints, lines in red along the ridges, thus separating the different drainage districts if more than one exist. A blue line through the valleys is of additional help in making district details of the topography.

17. District on Low Ground.—If a drainage district is very level and at a very slight elevation above tide water or above the river into which the sewage is to be discharged, the discharge sewer must be constructed to the lowest available point of outfall. This will then be the chief controlling condition, and it may even be necessary to resort to pumping in order to provide sufficient fall to cause the sewage to flow to the outlet. In such cases, the sewers empty into tanks, located at the lowest points, and their contents are then pumped to a sufficient height to flow to the outlet. Such conditions will often control the depth at which sewers can be located. For the purpose of removing the storm water, it is not generally necessary to locate the drains at any great depth below the surface; but, for the purpose of house sewers, it is necessary to locate them deep enough to collect sewage from the house fixtures, some of which may be in the basements.

18. Location of Sewer Lines.—The routes of the sewers should generally follow the natural drainage channels, as in this manner the best grades will usually be obtained. As the route of each sewer is decided on, it should be marked on the map in pencil. Profiles of the streets should also be made, as aids in determining the proper grades and depths for the sewers. It will be convenient, in deciding on the depth of the sewers, to draw on the profiles the depth and location of basements. The grade lines should be drawn in pencil on these profiles; after the final adjustment of the grade lines of the entire system, they may be inked in.

GRADES

19. Establishment of Grades.—The systems of main and lateral sewers should be so planned as to conduct the sewage to the outfall by the best available route. The most direct route in each case is generally the best route. In some cases, where the grades are very steep, and where, by adopting a longer circuit, the sewers may still have a sufficient grade and may be more nearly parallel to the surface at a uniform depth, the longer route may be the better location. The available grades are governed largely by the general slope of the surface, to which they must to some extent and in a general way conform. The sewers should be laid with fairly uniform grades, however, and should not follow too closely the variations in the surface.

Where the surface of the city or district is generally level, so that the grades are very flat, the main sewers must often be laid by the most direct and shortest routes in order to secure a sufficient inclination, even though the depth of excavation at some points is greater than might be necessary over other routes.

There are several controlling points in fixing grades and depths. Thus, the upper portions of lateral sewers should, if possible, be at depths sufficient to accommodate the houses along the lines. At a certain point, there may be a deep basement that must be provided for; at another point, a branch main may have to be brought into the system at a

given elevation. At still another point, the main sewer may have to be kept above a certain line, if possible, on account of quicksand and water, or other difficulties in construction. Above all, the elevation of the outlet must be such that the sewage may discharge freely at all times.

It is well to mark such points as these on the profiles in pencil, and then to harmonize, as much as possible, the requirements that may be conflicting, remembering always that gradients should not be below the necessary minimum to make the sewers self-cleansing.

TABLE I

MINIMUM GRADES FOR PIPE SEWERS

Diameter of Sewer Inches	Inclination When Depth of Flow Equals One-Half the Diameter		Velocity, Feet per Second
	Fractional	Per 100	
6	1 in 200	.5000	2.45
8	1 in 280	.3571	2.40
9	1 in 320	.3125	2.36
10	1 in 360	.2777	2.35
12	1 in 450	.2222	2.30
15	1 in 600	.1666	2.20
18	1 in 760	.1315	2.15
20	1 in 890	.1123	2.10
24	1 in 1,160	.0862	2.00
36	1 in 1,736	.0576	1.95
48	1 in 2,770	.0361	1.90
60	1 in 3,861	.0259	1.85
72	1 in 6,711	.0149	1.80
84	1 in 8,333	.0120	1.75
96	1 in 9,255	.0108	1.70
108	1 in 10,363	.0096	1.65
120	1 in 11,508	.0087	1.60

20. Minimum Grades.—The grades given in Table I may be taken as reasonable minimum grades for sewers

under ordinary conditions. Steeper grades are desirable at dead ends, and sometimes flatter grades must be adopted than are shown in this table. The velocities given in the fourth column are those corresponding to the given grades when the sewer is flowing half full. Flatter grades than those here given will not create enough velocity to keep the sewer free from sediment.

21. Loss of Grade Due to Changes in Size.—Where one sewer joins another of different size, or where the size of the sewer is changed, considerations of proper ventilation require that the inclination of the sewer shall be continuous

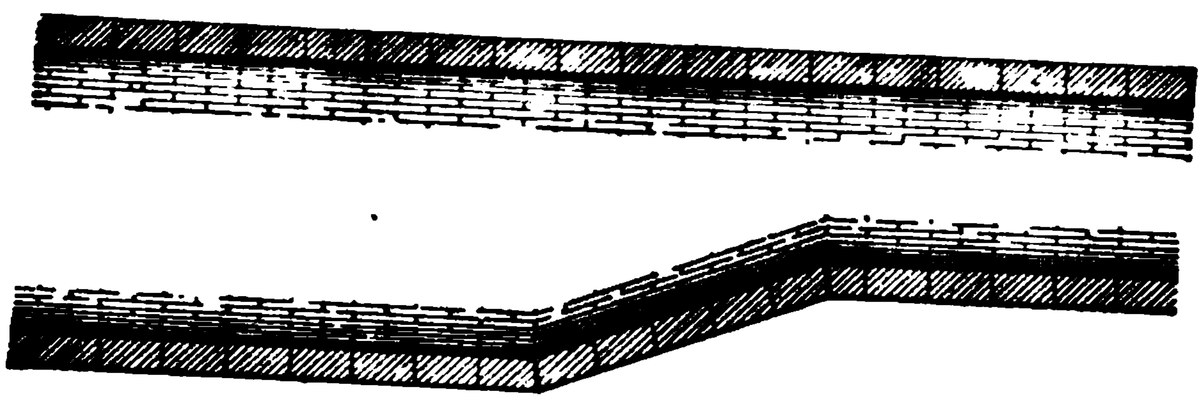


FIG. 2

along the **crown**, or upper part, in order not to obstruct the upward passage of the air-currents. Hence, the variation giving the change in size must be made wholly in the **invert**, or lower part, as indicated in Fig. 2. This will usually cause considerable loss of the available fall, and allowance must be

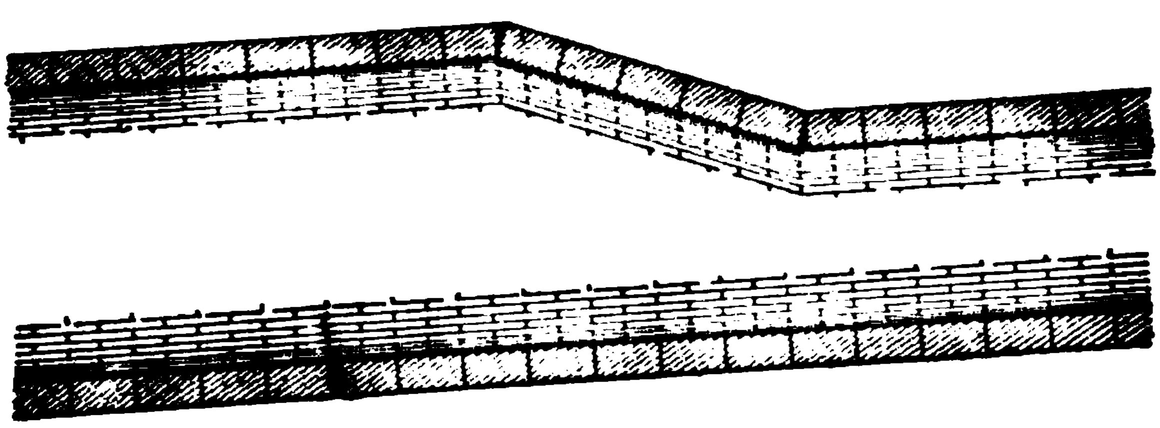


FIG. 3

made for such loss in determining the grades. The loss of the available fall will be not less than the difference between the diameters of the two sizes of sewer.

It will be noticed that, when the change of size is made in the manner shown in Fig. 2, an unobstructed passage is

afforded, not only to the downward flow of the sewage, but also to the upward flow of air. In order to have no loss of grade, where the available fall is small, the change of size is sometimes made as indicated in Fig. 3. This method, although giving a free passage to the downward flow of sewage, presents a material obstruction to the upward air-current, and is to be avoided when possible.

22. Depth of Sewer.—The depth at which a sewer should be located below the surface of the street will be determined principally by local conditions. Economical considerations would place the sewers as near the surface of the street as practicable, and storm sewers are usually shallow; other considerations, however, make it desirable that sometimes storm sewers—and often house sewers—should be located at considerable depth. Care should be taken to make the depth of sewer greater than the depth of the frost line. In order to avoid the water and gas mains and the surface pipes leading from them, which are necessarily placed below the frost line, it is advisable to locate the storm sewers just above these lines and the house sewers below them. The minimum depth of a house sewer, however, is generally determined by the positions of the house connections.

RAINFALL AND RUN-OFF

RATE OF RAINFALL

GENERAL CONSIDERATIONS

23. Important Condition.—In determining the required capacity for a storm-water sewer, one of the most important conditions to be considered is the maximum rate of rainfall; that is, the maximum rate of precipitation for any given number of minutes. A knowledge of this condition serves to determine the amount of storm water reaching the sewer during a storm continuing for a period of time sufficient to fill the sewer.

24. Records of Rainfall.—The records of rainfall are rather incomplete. Records of daily, monthly, and yearly rainfall are numerous; but useful as such records are for some purposes, they have little value for the design of sewers. The records of storms, as generally reported, give the total precipitation for the entire storm, and, possibly, the duration of the storm, but they do not usually give the *maximum rate* of the precipitation. The average rate of precipitation throughout the storm can be obtained by dividing the total precipitation by the duration of the storm. But this will seldom, if ever, be the maximum rate of precipitation; for, as is well known, the greatest intensity of the rainfall is attained only during short periods. It is often the case that a rainstorm will continue through several hours with a very uneven intensity, being sometimes a mere drizzle and sometimes a heavy downpour. Evidently, the total precipitation of such a storm will bear no relation to its maximum rate. A storm that will give 6 inches of rain in 12 hours may give 3 inches in 2 hours or, possibly, 2 inches in 30 minutes. The average precipitation during the entire

storm would then be at the rate of $\frac{6}{12}$, or $\frac{1}{2}$, inch per hour, while the precipitation during the 30 minutes of maximum rainfall would be at the rate of $2 \div \frac{1}{2}$, or 4, inches per hour.

25. Chief Condition to be Considered.—A locality subject to long-continued drizzling rains may have a large annual rainfall, while short, heavy rains may occur in localities having a much smaller annual rainfall. It is generally this maximum rate, or rapid downpour, during a reasonably short period, that most severely taxes the capacity of a storm-water sewer. The chief condition to be considered in designing a sewer of this kind is the maximum intensity of the precipitation; that is, the maximum rate per second or per hour, during a period of time sufficient for the water from the most remote parts of the district to reach the sewer and flow to the point under consideration.

26. Self-Registering Rain Gauges.—It is therefore evident that, in order to design intelligently a storm-water sewer, the designer should have a reasonably accurate record of the rainfall in the locality, giving both the rate and the duration of the varying degrees of precipitation for each storm. Such records are obtainable by means of self-registering rain gauges, in which the continuous amount of rainfall and the time are automatically recorded on a sheet moved by clockwork. In 1889, the United States Weather Bureau placed self-registering rain gauges in the principal American cities.

27. Valuable Data.—Much valuable information relating to the rainfall is given in the Weather Review, the official publication of the United States Weather Bureau. Only the records of self-registering gauges, however, can be considered as really accurate. On this subject, a great many useful data relating to conditions in the United States were also collected and compiled by the Board of Sanitary Engineers appointed by the president in 1889 to report on the sewerage of the District of Columbia. Diagrams of the rainfall were constructed by this board, by plotting on cross-section paper the rates per hour of excessive rainfall for

storms of different durations, as obtained from records of the precipitation in five of the principal cities of the United States. An inspection of such diagrams gives a comprehensive idea of the relative rates of rainfall for storms of different durations.

28. Uniformity in the Rate of Precipitation.—One important condition indicated by the diagrams just referred to is that the *maximum rate of precipitation for a given short period of time is reasonably uniform throughout the United States*. This is quite contrary to what has been the commonly accepted opinion. While the total amount of annual rainfall varies greatly in different parts of the United States, the greatest amount of rain that falls in a given short period of time does not appear to vary greatly in different parts of the country, although the maximum rates of rainfall are generally somewhat greater for the Southern coast states than for the interior. The frequency, however, with which the maximum rate of precipitation may be attained varies greatly throughout the country, as does also the total amount of rain that may fall during a single storm or during a season.

FORMULAS FOR RATE OF RAINFALL

29. General Equation for the Maximum Rate of Rainfall.—The maximum rates of rainfall given by storms of varying durations may be approximately expressed thus:

$$y = \frac{a}{x + b} \qquad (1)$$

in which a and b are constants;

y = rate of rainfall, in inches per hour, during a period of x hours.

The duration x of the storm may be expressed in minutes by multiplying both numerator and denominator of formula 1 by 60, which gives

$$y = \frac{60\,a}{60\,x + 60\,b} = \frac{a'}{x' + b'} \qquad (2)$$

in which

$$a' = 60\,a;$$
$$b' = 60\,b;$$
$$x' = 60\,x.$$

Here, x' is the duration of the storm in *minutes*, while y remains the rate of rainfall in inches per *hour*.

Likewise, the duration x of the storm may be expressed in seconds by multiplying both numerator and denominator of formula **1** by 3,600, which gives

$$y = \frac{3{,}600\,a}{3{,}600\,x + 3{,}600\,b} = \frac{a''}{x'' + b''} \qquad (3)$$

in which
$$a'' = 3{,}600\,a;$$
$$b'' = 3{,}600\,b;$$
$$x'' = 3{,}600\,x.$$

Here, x'' is the duration of the storm in *seconds*, while y remains the rate of rainfall in inches per *hour*.

30. Talbot's Formulas for Rainfall.—If, in formula **1**, Art. **29**, the values 1.75 and .25 are given to a and b, respectively, the result will be the formula proposed by Professor Talbot, of the University of Illinois, for the maximum rate of ordinary rainfall:

$$y = \frac{1.75}{x + .25} \qquad (1)$$

If, in the same formula, values of 6 and .5 are given to a and b, respectively, the result will be Talbot's formula for the maximum rate of rare rainfall:

$$y = \frac{6}{x + .5} \qquad (2)$$

Formula **2**, however, is of no practical value in the design of storm-water sewers; because, while it gives a correct maximum rate of rainfall, the size of sewer corresponding is so large as to make the expense prohibitive. It is wiser to let a sewer be overtaxed once in 10 years or so than to have it decidedly too large for the rest of that period.

According to Professor Talbot, it is probable that storms reaching values given by his formula for the maximum rate of ordinary rainfall will occur at a given point two or three times in 10 years, and the values given by his formula for rare rainfall will not be exceeded oftener than once in 50 years or more. It must be noticed, however, that the frequency with which the values given by formula **1** will be

likely to be reached or exceeded by actual storms will vary considerably in different parts of the United States. Storms attaining the rate of precipitation given by this formula may be expected to occur much more frequently in the southern Atlantic states than in the middle or western states. The values given by Talbot's formula for ordinary rainfall apply reasonably well throughout most of the northern and western states.

31. Other Formulas for Rainfall.—As a basis for the design of sewers in some localities, it will be advisable to use a formula giving somewhat higher rates of precipitation than those given by Talbot's formula. For localities in which the rainfall is frequent and heavy, values of 2.25 and .3, substituted in formula **1**, Art. **29**, for a and b, respectively, appear to be satisfactory. This gives the following formula for the maximum rate of occasional rainfall in such localities:

$$y = \frac{2.25}{x + .3} \qquad (1)$$

Rates of rainfall given by this formula will probably not be exceeded at any given point in the United States oftener than once in about 5 years. The formula, therefore, appears to be satisfactory for the maximum rate of occasional rainfall in localities where rainstorms are of frequent occurrence, as in the southern Atlantic states; it will be used in the Examples for Practice given in this Section to express the maximum rate of rainfall attained by occasional storms, although it is really no more general in its application than formula **1**, Art. **30**.

It is probable that, for some parts of the United States, neither formula **1**, Art. **30**, nor formula **1** of this article will satisfactorily express the maximum rate of rainfall to be used in designing storm-water sewers. In such cases, a formula suitable to the locality must be derived from the records of rainfall in that region. The records of rainfall should be obtained from self-registering rain gauges. The formula for rainfall may be obtained for any locality by

substituting suitable values for a and b in formula **1**, Art. **29**.

An expression for the maximum rate of occasional rainfall that is better adapted to some localities than formula **1** may be obtained by using the values 2 and .25 for a and b, respectively; this gives

$$y = \frac{2.00}{x + .25} \qquad (2)$$

It should be noticed that all the foregoing formulas express the rate per hour and not the total precipitation.

32. Violent Storms.—In cases of heavy downpours, a small quantity of the storm water can be conveyed in the surface gutters, thus relieving the sewers to some extent; hence, no serious damage will usually result if, at rare intervals, the rate of precipitation is such as to give a flow somewhat in excess of the actual capacities of the sewers. It is not customary, nor is it generally considered necessary or even desirable, to design sewerage systems with capacities sufficient for the prompt removal by means of the sewers alone of the entire rainfall from exceptionally heavy storms. Such storms occur only at long intervals and are of short duration. Further, sewers with capacities sufficient to meet the conditions of excessive storms would not be advantageous as conduits for the ordinary flow. The values given by formula **1**, Art. **31**, appear to be adequate for the purpose of estimating the rates of rainfall in designing storm-water sewers under the most extreme conditions. Values given by formula **2**, Art. **31**, apply to less severe conditions; while those given by formula **1**, Art. **30**, apply satisfactorily to the more ordinary conditions.

EXAMPLE.—What is the maximum rate per hour of ordinary rainfall, according to Talbot's formula, given by a storm of 30 minutes' duration?

SOLUTION.—Writing formula **1**, Art. **30**, in the form of formula **2**, Art. **29**, and substituting the number of minutes of the duration of the storm for x', there results

$$y = \frac{60 \times 1.75}{30 + 60 \times .25} = \frac{105}{45} = 2.33 \text{ in. per hr.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. What is the maximum rate per hour of precipitation that may occasionally be attained by ordinary storms of 30 minutes' duration? Ans. 2.81 in.

2. What is the maximum rate per hour of an occasional rainfall for a storm of 1 hour and 30 minutes' duration? Ans. 1.25 in.

3. What may be considered the maximum rate of precipitation occasionally attained by storms of 10 minutes' duration? Ans. 4.82 in.

4. What is the maximum rate per hour of an occasional rainfall as given by a storm of 50 minutes' duration? Ans. 1.99 in.

---

33. Tabulated Rates of Rainfall.—A satisfactory formula for the rate of rainfall for any particular locality having been established, the values of y, the rate of rainfall, may be readily tabulated for convenient use. In Table II are given the values of y for values of x varying from 2 minutes to 5 hours, as given by formula **1**, Art. **31**. For values of x intermediate between those given, values of y may be interpolated.

34. Rainfall in Cubic Feet per Second per Acre.—It will be well here to note that the rate of rainfall in inches per hour corresponds very closely to the number of cubic feet per second falling on 1 acre, the difference being less than 1 per cent. This is readily shown as follows: There are 43,560 square feet in an acre. A rainfall of 1 inch ($= \frac{1}{12}$ foot) per hour would give $\frac{1}{12} \times 43{,}560 = 3{,}630$ cubic feet per hour per acre, or

$$\frac{3{,}630}{60 \times 60} = \frac{121}{120} = 1.0083 \text{ cubic feet per second per acre}$$

An assumed rate of 12 inches, or 1 foot, per hour would give 43,560 cubic feet per hour per acre, or $\frac{43{,}560}{60 \times 60} = 121$ cubic feet per second per acre. Therefore, in all computations relating to the required capacities of sewers, the number of cubic feet per acre falling in 1 second may, without material error, be taken to be the same as the number of inches in depth falling per hour.

TABLE II

MAXIMUM RATES OF RAINFALL AS GIVEN BY FORMULA 1, ART. 81

Duration of Storm			Rate of Rainfall	Duration of Storm			Rate of Rainfall	Duration of Storm			Rate of Rainfall
Hours x	Minutes x'	Seconds x''	Inches per Hour y or y_1	Hours x	Minutes x'	Seconds x''	Inches per Hour y or y_1	Hours x	Minutes x'	Seconds x''	Inches per Hour y or y_1
	2	120	6.75		35	2,100	2.55		100	6,000	1.14
	5	300	5.87		40	2,400	2.33		110	6,600	1.05
	8	480	5.19	.75	45	2,700	2.14	2.00	120	7,200	.98
	10	600	4.82		50	3,000	1.99		130	7,800	.91
	12	720	4.50		55	3,300	1.85		140	8,400	.85
.25	15	900	4.09	1.00	60	3,600	1.73	2.50	150	9,000	.80
	18	1,080	3.75		65	3,900	1.63	2.75	165	9,900	.74
	20	1,200	3.55		70	4,200	1.53	3.00	180	10,800	.68
	22	1,320	3.38	1.25	75	4,500	1.45	3.50	210	12,600	.59
	25	1,500	3.14		80	4,800	1.38	4.00	240	14,400	.52
	28	1,680	2.93		85	5,100	1.31	4.50	270	16,200	.47
.50	30	1,800	2.81	1.50	90	5,400	1.25	5.00	300	18,000	.42

The number of cubic feet of rain per second falling on 1 acre will hereafter be designated by y_1. For any duration of storm, the value of y_1, according to formula **1**, Art. **31**, will be taken from Table II, or interpolated between the values there given.

35. Rate for Purposes of Design.—The preceding paragraphs show the relation that exists between the duration of a storm and the rate at which the rain falls per hour. The size of sewer, as will be seen later, depends directly on the rate of rainfall, which in turn depends on the length of the storm chosen. When the proper duration of the storm has been selected, the rate is determined by one of the formulas; but it is most important to note that the rate, after choice of the formula, depends directly and entirely on the proper selection of the duration.

36. Choice of Proper Duration of Maximum Storms.—If a district is large and the surface comparatively level, the motion of the rainwater reaching the ground is slow. The water, therefore, that falls near the outlet has time in a short storm to run off before the water from the farthest point of the district reaches that point. It is generally assumed that the maximum effect on the volume of the run-off takes place when the storm lasts just long enough for the water from the farthest point of the district to reach the outfall. A storm lasting longer than this has a lower rate, and, therefore, will not bring so large a volume to the outlet. A shorter storm gives a higher rate, but the storm is over before the water from distant points has reached the outlet. The capacity of the sewer is also affected by the shape of the district, as well as by the slope and character of the surface.

The flow of the storm water over the surface of a flat district may require double the time required for the flow over a district of the same size having a sloping surface. The flow will be much more rapid over the surface of a paved district than over the surface of an unpaved district, and more rapid over a smooth lawn than over a wooded tract.

If, to reach the sewer, the water must flow through rough and crooked channels, it will require much longer than if led by direct courses through smooth conduits.

The shorter the storm, the higher will be the rate of rainfall; but, in order to give the highest rate of flow entering the sewer, the storm must continue until the water from the most remote parts of the district begins to flow into the sewer.

37. Condition Producing Maximum Flow.—From what has been said, it is evident that *the rate of rainfall that will produce the greatest flow in the sewer at any given point will be the maximum rate that will continue (after the ground has become saturated) for a length of time sufficient for the water from the most remote parts of the district to flow to the sewer and down through the sewer to the point under consideration; that is, to any point at which the size of the sewer is to be determined.* Hence, having ascertained the length of time required for this condition, as will be explained further on, it is easy to determine the maximum rate of rainfall for which to provide. Thus, if it requires 10 minutes for the storm water from the most remote parts of the district to reach the point where the size of the sewer is to be determined, the rate of rainfall y that will cause the maximum flow in the sewer, as given by Table II, will be 4.82 inches per hour, or 4.82 cubic feet per second per acre. The high rate of precipitation in this case is due to the fact that the storm water reaches the given point so promptly that a storm of short duration will give the maximum flow.

PROPORTION OF RAINFALL REACHING SEWER

38. General Statement.—In the preceding articles, formulas were given for estimating the rate of rainfall. It will now be necessary to notice the proportion of the rainfall that will reach the sewer during the period of maximum flow. It should be borne in mind that *only a fraction of the total rainfall will reach the sewer*, and a still smaller fraction will

reach the sewer during the period of greatest flow, which is the most essential condition to be considered in determining the capacities required for storm-water sewers.

39. Conditions Affecting Flow of Storm Water. The proportion of the rainfall that reaches the sewer varies according to the area, slope, and condition of the surface, and the nature of the subsoil. Wooded tracts, cultivated lands, or those covered by luxuriant vegetation retain a greater part of the rainfall, and are longer in yielding up what they do not retain, than smoothly cut lawns or areas devoid of vegetable growth. The latter will, therefore, give the greater flow. The amount of the flow is also affected by the nature of the soil. Loose, porous soils, as sand or loam, readily soak in a large proportion of the rainfall, while hard-packed and impervious soils, as clay and cemented material, take in much less of the rainfall and give much greater surface flows. Steep slopes throw off a much greater proportion of the rainfall than flat areas, and carry it more quickly to the channels of flow. Hence, a hilly country will not only yield a greater proportion of the storm water than a level country, but will also deliver it to the sewers much more quickly. Frozen ground may give a surface flow practically equal to the rainfall; and, if the rain occurs simultaneously with the melting of snow, the surface flow may considerably exceed the rainfall.

40. Flow of Storm Water From Built-Up Districts. The conditions already noticed refer principally to suburban districts. In districts that are closely built up and have paved streets, the proportion of storm water carried to the sewers and the rapidity with which it will be conveyed to them are both greatly augmented. The greater part (often the whole) of the surface on which the rain falls consists of paved streets and courts, walks, and the roofs of buildings, all of which offer nearly impermeable surfaces to the rainfall, while the systems of surface drains, troughs, and gutters quickly convey it to the sewers. As a result, a large fraction of the rainfall is promptly delivered to the sewers,

which will be severely charged by short storms having a high rate of precipitation.

41. Ratio of Storm Water to Rainfall Found to be Constant.—Gaugings of sewers during the flow of storm water indicate that, after the ground has become saturated, the ratio of storm water to rainfall is practically constant for a given district; that is, after saturation, the percentage of rainfall reaching the sewer is practically the same for all rates of precipitation. In the case of roofs and well-paved areas, the effect of saturation is slight. As already noticed, the ratio of the storm water to the total rainfall depends largely on the character and condition of both the surface and the subsoil. Gaugings sufficient to establish this ratio definitely for different conditions of surface have never been made. Such reliable information as is available concerning the subject will be briefly noted.

42. E. Kuichling, of Rochester, New York, a well-known hydraulic engineer and an eminent authority on storm-water sewerage, gives the percentages in Table III as representing

TABLE III

RELATION OF FULLY IMPERVIOUS SURFACE TO TOTAL AREA ACCORDING TO DENSITY OF POPULATION

Class	Average Number of Persons per Acre	Percentage of Fully Impervious Surface			
		Roofs	Improved Streets	Unimproved Streets, Yards, Etc.	Total
	15	8.4	3.3	3.0	14.7
c	25	14.0	7.0	4.3	25.3
d	32	18.0	10.2	5.0	33.2
e	40	22.5	14.7	5.4	42.6
f	50	28.0	19.0	5.6	52.6

the relations of the impervious surface to the total drainage area, assuming these relations to vary according to the density of population. These values were obtained from an extended analysis of the conditions found in such cities as

Buffalo, Rochester, and Syracuse. The percentages in the last column should be used as the ratios of storm water reaching the sewer to total precipitation, since, that per cent. of the area being impervious, the water on that per cent. will all be directly delivered to the sewer.

43. Contemporary Flow.—The percentages already given relate to the total amount of storm water flowing from a given storm. What is most important to obtain in designing storm-water sewers, however, is the ratio of the **contemporary flow** to the rainfall; that is, the proportion of the rainfall that will enter the sewer as storm water during a period of time equal to the duration of the storm. This condition will generally produce the maximum flow in the sewer. Of the total amount of storm water flowing to the sewer from a given storm, not more than from one-quarter to three-quarters of it will generally reach the sewer during the continuance of the storm. This ratio cannot be stated with any great degree of accuracy.

For cities, Professor Talbot gives the following as the ratio of storm water to contemporary rainfall:

Class	Character of District	Ratio
b	Suburban districts, sewered but not paved .	.20
d	Suburban districts, paved and sewered . . .	.30 to .40
f	Closely built-up districts, paved and sewered	.40 to .50
h	Roofs of buildings	Nearly 1.00

Assuming the condition of the surface to vary according to the density of the population, he also gives the following ratios as indicating the proportion of the rainfall flowing off at once:

Class	Population per Acre	Ratio
a	10	.10
b	20	.20
e	40	.30
f	50	.40
g	Denser	.50 or more.

For convenience of comparison, corresponding items of the ratios and percentages given in this and other articles are designated by the same letter.

MAXIMUM RATE OF FLOW

44. London Gaugings: Rate of Flow.—The period of maximum flow may be assumed to be the same as the time of the duration of the storm, although it will generally require a considerably longer time to carry off the total amount of storm water given by the storm. Of the available data relating to this phase of the subject, probably the most reliable are those obtained from gaugings of sewers in London, England. From these gaugings, the length of time required for the sewers to carry off the rainfall was found to be from three to four times the duration of the storm; or, in other words, the duration of the storm was found to be from one-fourth to one-third of the time required for the storm water to flow away. But it was also found that the quantity of storm water reaching the sewer during different periods of this time varied greatly, being for certain short periods as high as 2.4 times the average storm flow; that is, the maximum flow per second was found to rise as high as 2.4 times the average flow per second from the entire storm. Therefore, in order to provide for this condition, the sewer capacity must be sufficient to carry, *during a length of time equal to the duration of the storm*, from $\frac{1}{4} \times 2.4 = .6$ to $\frac{1}{3} \times 2.4 = .8$ of the total amount of storm water given to the sewer by the storm.

45. Ratio of Storm Flow to Rainfall.—In different districts of London, the ratio of the total amount of storm water to the total amount of rainfall was found by the gaugings to vary from .53 to .94. As London is one of the most densely populated and completely paved cities in the world, it is probable that the maximum ratio of .94 was for districts consisting of solid stretches of roofs, walks, and pavements in perfect condition, and may be taken to represent the absolute maximum of the total storm-water flow. For this condition, the flow of storm water during a length of time equal to the duration of the storm will be from $.6 \times .94 = .56$ to $.8 \times .94 = .75$ of the rainfall. It is to be remembered that

these ratios are for the most extreme conditions of a very densely populated city, and that they will not apply to ordinary cities.

For the minimum ratio of .53, the corresponding flow of storm water will be from $.6 \times .53 = .32$ to $.8 \times .53 = .42$ of the rainfall. This ratio will be assumed to have been for an ordinary district, rather closely built up and ordinarily paved, such as might exist in the outlying portions of London; the population will be assumed as 50 per acre. By applying the same ratio of the contemporary to the total flow of storm water to Kuichling's percentage of impervious ground, a ratio of from $.6 \times .526 = .32$ to $.8 \times .526 = .42$ is obtained for a population of 50 per acre; this agrees well with the foregoing.

46. Coefficient of Storm Flow.—The ratios thus obtained represent, for the given conditions, the proportion of the rainfall to be assumed as flowing off during a period of time equal to the duration of the storm. This proportion of the rainfall will probably not flow away during this period of time, but the rate of flow given by this assumption will be such as may be attained by the storm water during short periods of its flow, and for which sewer capacity must be provided. The rate of flow thus obtained will, however, represent the extreme conditions. This ratio will hereafter be called the **coefficient of storm flow**, and will be designated by f. The ratio f is so affected by many and various local conditions that it is impossible to state its value accurately for districts of different character. Taking the foregoing figures, however, in connection with Kuichling's percentage of impervious ground, as a basis, the approximate values of f given in Table IV may be taken as a general guide. A good designer should take every opportunity to compare rainfall rates with flood-flow measurements with which he is familiar, and so acquire a definite conception of the coefficient of storm flow that is appropriate for different kinds of districts.

In Table IV, the various districts of different character are, for convenience of reference, designated by letters.

The letters designating the various classes correspond as nearly as possible to those of the ratios and percentages previously given. There is no well-defined distinction between the districts of different character; they merge into each other. Judgment and care must be exercised in selecting the class to which any given district belongs. The values of f given in the table relate to the maximum rates of flow

TABLE IV

RATIO f, OR COEFFICIENT OF STORM FLOW, FOR DRAINAGE DISTRICTS

Class	Population per Acre	Character of District	Values of f			Value of f_1
			Minimum	Maximum	Mean	
a	10	Unimproved suburbs . . .	.06	.12	.09	
b	20	Improved suburbs, unpaved	.10	.18	.14	.31
c	25	Macadamized residence suburbs	.15	.21	.18	.375
d	32	Ordinary suburban districts, roughly paved . .	.20	.28	.24	.44
e	40	Built-up, paved districts	.26	.34	.30	.52
f	50	Closely built-up and well-paved districts	.32	.42	.36	.625
g		Densely built-up and exceedingly well-paved districts	.42	.56	.50	.75
h		Solid stretches of roofs, walks, and pavements, in perfect condition . .	.55	.75	.65	

to which the storm water may rise. As this maximum flow will be maintained only during comparatively short periods of time, it follows that, when used with reference to a period of time equal to the duration of the storm, it will generally be sufficient to use the minimum values of f for large districts. The mean values of f will be used here, however. The values f_1 given in the last column of the table will be explained further on.

47. Conditions Affecting Ratio of Storm Flow. The quantity of storm water represented by the ratios given in Table IV may be assumed to reach the sewer during a period of time equal to the duration of the storm. The ratio will, of course, vary somewhat, according to the size of the district and the slope of the surface. If the district is large and its surface comparatively level, considerable time will be required for the storm water to flow over it to the sewer, and opportunity will be afforded for a large amount of evaporation and absorption; hence, the percentage of the rainfall reaching the sewer will be small. Moreover, in large districts, the maximum rate of precipitation given by the storm will not always extend throughout the entire district. During storms of long duration, the flow of the storm water from such districts will gradually increase, until the water from the most remote parts of the district reaches the sewer. If the storm is of short duration, it may cease before the water from remote parts of the district begins to enter the sewer. It is often the case in large districts that the greatest flow given by a storm of short duration occurs after the storm has abated. If, on the other hand, the district is small and its surface very sloping, the storm water will quickly reach the sewer, and the proportion evaporated and absorbed will be small. In such a district, the heavy flow of storm water will begin during the early stages of the storm and will rapidly decrease soon after the storm abates. These conditions affect to some extent the amount of storm water given to the sewer, and must be considered in using the ratios given in Table IV. In general, it may be stated that the maximum values of f should be used for small districts having very sloping surfaces, and the minimum values for large and comparatively level districts.

RATIONAL FORMULA FOR RATE OF FLOW

48. General Expression for Contemporary Flow. From the conditions stated, taken in connection with the formula for the rate of maximum rainfall, may be derived a

rational formula for determining the rate of flow per second that must be provided for in designing a storm-water sewer.

As y_1 represents the number of cubic feet of rainfall per second per acre (taken equal to the rate of rainfall in inches per hour), if the duration x of the storm is expressed in seconds, the value of y_1 will be given in the column y_1 of Table II opposite the value of x'' given in the column x''. If, now, f represents the ratio of the storm water to contemporary rainfall, as given by Table IV, then fy_1 will represent the contemporary flow per acre, or the number of cubic feet of storm water per acre, reaching the sewer each second during a period of time equal to the duration of the storm. Substituting y_1 for its equivalent value y in formula **3**, Art. **29**, and multiplying both terms of the equation by f, there results

$$fy_1 = f \times \frac{a''}{x'' + b''}$$

49. Derived Flow per Acre.—If F is taken to represent the flow per acre of storm water entering the sewer, in cubic feet per second, then

$$F = fy_1 \qquad (1)$$

From the formula of the preceding article we may write

$$F = f \times \frac{a''}{x'' + b''}$$

Writing the values of a'' and b'' from formula **3**, Art. **29**,

$$F = f \times \frac{3{,}600\,a}{x'' + 3{,}600\,f}$$

Substituting the values of a and b from formula **1**, Art. **31**,

$$F = f \times \frac{2.25 \times 3{,}600}{x'' + .3 \times 3{,}600} = f \times \frac{8{,}100}{x'' + 1{,}080} \qquad (2)$$

This formula may be readily solved by substituting the values of f and x''. But, for any value of x'', the value of the expression $\frac{8{,}100}{x'' + 1{,}080}$, or y_1, may be taken directly from Table II. Hence, for a storm of any duration x'', the value of F, in cubic feet per second, will be given by formula **1** of this article, by taking the value of f from Table IV and the value of y_1 from Table II.

50. Flow at Inlet.—If t is taken to represent the length of time, in seconds, required for the storm water from the most remote parts of the district to reach the inlet of the sewer, then, for the maximum flow of the sewer, at the inlet, t will equal x'' of formula **3**, Art. **29**, and may be substituted for x'' in formula **2**, Art. **49**, giving for this condition the formula

$$F = \frac{8{,}100\,f}{t + 1{,}080}$$

This formula determines the required capacity of the sewer in cubic feet per second at its upper inlet.

51. Flow at Points Below Inlet.—For determining the required capacity of the sewer at any given point below the inlet, the rate of flow along the sewer must also be taken into consideration. This will vary with the grade and with the character and size of the conduit; it may be determined by applying the ordinary hydraulic formulas, which will be considered further on.

Let l = length of sewer, in feet, from inlet to point under consideration;

v = average velocity of flow in sewer, in feet per second.

Then $\frac{l}{v}$ = time, in seconds, required for water to flow from inlet to point under consideration.

$t + \frac{l}{v}$ = total time, in seconds, for water to flow from most remote parts of district to point under consideration.

Now, $t + \frac{l}{v}$ is the duration of that storm that will give the maximum flow of storm water at the point under consideration (see Art. **37**). The substitution of this value of x'' in formula **2**, Art. **49**, gives, for this condition,

$$F = \frac{8{,}100\,f}{t + \frac{l}{v} + 1{,}080}$$

This is the most rational form of equation for the required capacities of storm-water sewers that has yet been proposed, though the numerical constants may not apply to all cases.

52. Talbot's Formula for the Rate of Flow.—Professor Talbot derived a formula having the same form as that given in the preceding article, from his formula for the maximum rate of ordinary rainfall; it is as follows, using the same notation as in the formula just given:

$$F = \frac{6{,}300\,f}{t + \frac{l}{v} + 900}$$

53. Objection to Element of Time in Formula. The objection made to the foregoing formulas is that they include the element of time. But, as the rate of rainfall varies greatly with the time of the duration of the storm, it seems impossible to derive a rational formula for this purpose that does not include the element of time. As the length of the sewer l, from the inlet to the point under consideration, is always known, and, by assuming dimensions, the velocity v may be calculated by the ordinary formulas of hydraulics, the value $\frac{l}{v}$ may be readily determined. The most difficult feature in applying the formula is to determine satisfactorily the length of time required for the storm water from the remote parts of the district to reach the sewer. This period of time t may be determined by experiment, or the length of open gutter may be estimated, and a velocity of from 1 to 3 feet per second assumed. An interval of about 5 minutes must also be assumed as the time necessary for the water to get into the gutter from the point where it falls on the ground. Having determined the value of t for one part of the district, then, for any other part, it may be determined by proportion, all other conditions being the same. If k is taken to represent the length, in feet, of the path traversed by the water in reaching the sewer, the time t may be taken directly proportional to k.

54. Surface Velocities for 1-Per-Cent. Slope.—The period of time t, in seconds, required for the storm water to reach the sewer may also be approximately estimated as follows: If v_s denotes the average velocity of the surface flow,

in feet per second, the time t will equal $\frac{k}{v_s}$. The value of k is known, and the value of v_s may be approximately determined. For an average surface slope of 1 per cent., that is, for a fall of 1 foot in 100 feet, the approximate effective surface velocities v_1, in feet per second, as estimated for different characters of surface, are given in Table V. The values given in this table do not represent actual velocities;

TABLE V

EFFECTIVE VELOCITIES v_1, IN FEET PER SECOND, FOR AN AVERAGE SURFACE SLOPE OF 1 PER CENT.

Class	Character of Surface	Velocities v_1		
		Minimum	Maximum	Mean
a'	Woodlands and heavy vegetable growth	.11	.22	.16
a	Pasture lands and meadows	.14	.28	.21
b	Smooth lawns	.19	.37	.28
c	Firm gravel and macadam	.28	.52	.40
d	Rough stone pavements	.45	.75	.60
e	Ordinary pavements	.60	1.10	.85
f	Good pavements	.75	1.45	1.10
g	Perfect pavements and gutters	.95	1.85	1.40
h	Asphalt pavements, perfectly paved gutters, roofs, troughs, and similar surface conduits	1.60	2.40	2.00

they represent the effective velocities, that is, the velocities that the water would need to have in order to reach the sewer in the same length of time that it actually does, if it flowed in a direct line.

Under the usual conditions, the water flows to the sewer by very devious courses and at considerable higher velocities than those given in the table, which are merely rough approximations. For convenience, these values will be used throughout this Section, but they are not to be relied on in practice. The surface velocities, or the time t, should, as far

as possible, be determined by experiment, or by computation as soon as the surface grades are known.

55. Surface Velocities for Other Slopes.—By the slope S, expressed as a per cent., is meant the fall of the surface, in feet, in each 100 feet in length of slope, or the sine of the angle of the average slope with the horizon, multiplied by 100; thus, if h is the total fall, in feet, along the path k, then

$$S = \frac{100\,h}{k}$$

Having obtained the surface velocity for a slope of 1 per cent., the surface velocity v_s for any other slope may be obtained by proportion: the velocities will be to each other as the square roots of the slopes, all other conditions being equal. Hence, if S is expressed as a per cent., we shall have the proportion

$$v_1 : \sqrt{1} = v_s : \sqrt{S},$$

from which $$v_s = v_1\sqrt{S}$$

Consequently, having the velocity v_1 for a slope of 1 per cent., to find the velocity v_s for any other slope, *multiply v_1 by the square root of the slope.* It is evident that the surface velocity will be affected by the form as well as by the character of the district.

56. Practical Formula for the Flow per Acre.—It will be noticed that the period of time t required for the water to flow to the sewer is equal to

$$\frac{k}{v_s} = \frac{k}{v_1\sqrt{S}}$$

Substituting the latter expression for t in the formula of Art. **51**, we have

$$F = \frac{8{,}100\,f}{\dfrac{k}{v_1\sqrt{S}} + \dfrac{l}{v} + 1{,}080} \qquad (1)$$

If, in order to meet special conditions, it is desired to use different values for the constants a'' and b'' in the equation for the rate of rainfall, such values may also be substituted

for a'' and b'' in the general form of this formula, which is as follows:

$$F = \frac{f a''}{\frac{k}{v_s} + \frac{l}{v} + b''} \qquad (2)$$

It will be noticed that formula 2 is simply the formula given in Art. 48 with the expression $\frac{k}{v_s} + \frac{l}{v}$ substituted for x'', and that formula 1 is formula 2 of Art. 49, with the expression $\frac{k}{v_1 \sqrt{S}} + \frac{l}{v}$ substituted for x''. Consequently, formula 1 may readily be solved by first finding the value of the expression $\frac{k}{v_1 \sqrt{S}} + \frac{l}{v}$, which is equal to x'', then taking from Table II the value of y_1 corresponding to this value for x'', and finally taking from Table IV the value of f.

Formula 1 will herein be used for determining the required capacities for storm-water sewers. As thus rationally derived, it is a practical and flexible formula for this purpose. By embracing all essential conditions, it employs all acquired data for determining the flow, and becomes a safe working formula, permitting the intelligent exercise of judgment and discretion in deciding the values of the various quantities. The values of all quantities should be determined as accurately as possible. The values of f and v_s should be determined by experiment, when possible; the correct values of k and l may be readily obtained.

57. The exact value of v may be calculated by methods that will be explained in a subsequent article. For preliminary calculations, however, v may generally be assumed to be as given in Table VI.

The designer always has a general idea of the size of sewer to be used; from that size and the grade, an appropriate value of v can be taken from the table. For convenience, the velocities given in this table will be used here in calculations. A slight error in the value of v will not greatly affect the resulting value of F. The value of F given

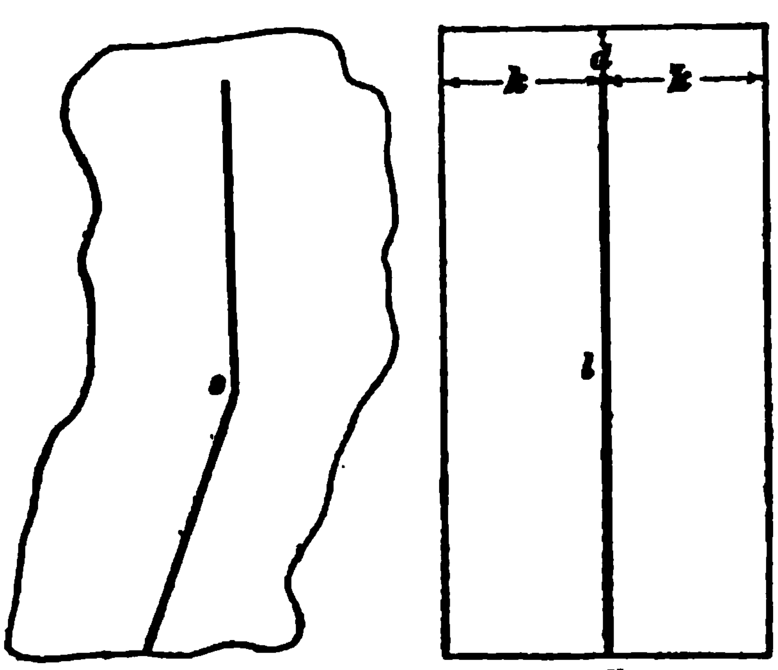
by formula **1** of Art. **56** will be the flow per acre in cubic feet per second, and must be multiplied by the number of acres drained in order to give the total flow per second to be provided for.

TABLE VI

APPROXIMATE VELOCITIES OF FLOW IN SEWER, IN FEET PER SECOND, TO BE ASSUMED FOR DIFFERENT GRADES

Class	Grade	Appearance	Diameter of Sewer Inches	Velocity Feet per Second	Diameter of Sewer Inches	Velocity Feet per Second
1	1:5,000	Very flat	18	$\frac{3}{4}$	72	2
2	1:1,000	Flat	18	2	72	4
3	1:500	Moderately flat	12	2	36	3.5
4	1:100	Ordinary	12	4	36	8
5	1:10	Steep	8	10	12	14
6	1:5	Very steep	8	17	12	30

58. Form of Drainage District.—Although drainage districts are usually somewhat irregular in form, most districts are approximately rectangular, and, for the purposes of estimating the storm-water flow, may generally be assumed to be rectangular, with the main trunk sewer extending longitudinally through the middle of the district. In Fig. 4, (*a*) may represent the actual form of a drainage district tributary to the main drainage sewer *s*. In computing the required capacity of the sewer, however, the form of the district may generally be assumed to be as shown at (*b*). In such a case, the length *l* of the sewer, below the upper inlet that receives water from a remote part of the



(*a*) (*b*)

Fig. 4

district, may be taken equal to the length of the district minus d, the distance from the inlet to the upper end of the district; while k will usually be equal to one-half the width of the district at its upper end, as shown.

59. Area of Drainage District.—There are 43,560 square feet in an acre. Hence, the number A of acres in the rectangular drainage district shown in Fig. 4 (b) will be given by the formula

$$A = \frac{2k(l+d)}{43,560} = \frac{k(l+d)}{21,780} \qquad (1)$$

in which k = one-half the width, in feet; $l + d$ = total length of drainage district, in feet.

It is evident that formula **1** will give approximately the number of acres in the drainage district shown in Fig. 4 (a).

The number of acres in a given district may usually be roughly approximated by the formula

$$A = \frac{kl}{21,000} \qquad (2)$$

If the flow in the sewer is to be determined at some point above the outlet, then, in the formulas of Arts. **51, 52, 56,** and **59,** l will be the distance from the inlet to the point under consideration.

60. The Total Effluent. As F is the flow per acre in cubic feet per second, the **effluent** E, or total flow from the district in cubic feet per second, will be equal to F multiplied by the number of acres, or $A F$. By multiplying together the corresponding terms of formula **1,** Art. **56,** and formula **1,** Art. **59,** and writing E for $A F$, the following formula is obtained:

$$E = \frac{k(l+d)}{21,780} \times \frac{8,100f}{\frac{k}{v_1\sqrt{S}} + \frac{l}{v} + 1,080}$$

in which E = total effluent, or flow from entire district, in cubic feet per second (equal to capacity required for sewer);

k = length of path over which water from most remote part of district must flow to sewer

inlet (assumed equal to one-half the width of district);

S = average slope of surface on path k, in feet per hundred;

l = length of sewer from inlet to point at which flow is to be determined;

d = distance from upper edge of district to inlet [Fig. 4 (*b*)];

f = coefficient of storm flow (Table IV);

v_1 = surface velocity for an average slope of 1 per cent. (Table V);

v = mean velocity in sewer (Table VI).

This is a general working formula for the required capacity of a storm-water sewer. It will, however, be well to note that, if the district is very irregular, the number of acres contained in it will not be correctly given by formula **1**, Art. **59**, which will also affect the accuracy of the formula given in this article. In such a case, the flow per acre F should be calculated by formula **1**, Art. **56**, and the result multiplied by the number of acres in the district, as calculated from measurement or other available information. The latter method is to be preferred.

It should also be noticed that the distance k, or path of the water flowing over the surface to the sewer inlet, which in Fig. 4 (*b*) is shown equal to one-half the width of the district, is not usually of nearly so great a length. The two lines marked k in the figure would, in most instances, represent the positions of branch sewers. In such cases, the length k of the path of the surface flow would usually be the distance from the upper end of the longer branch to the upper margin of the district, which would generally be equal to the distance d. For simplicity in the problems, the path k of the surface flow is assumed as shown in Fig. 4 (*b*).

EXAMPLE.—In a rectangular suburban drainage district, roughly paved with stone pavement (class *d*), having an average slope toward the sewer of 2 feet in 100, and in which the distances k, l, and d are 660, 5,000, and 280 feet, respectively, what will be the required capacity, in cubic feet per second, at the lower end of a storm-water sewer

draining the district, assuming a value of 4 feet per second for v, and using for f and v_1 the mean values given for class d in Tables IV and V?

SOLUTION.—The mean value of f, as given for class d in Table IV, is .24, while from Table V, the mean value of v_1 for the same class is .60. Hence, by the formula of this article,

$$E = \frac{660 \times (5{,}000 + 280)}{21{,}780} \times \frac{8{,}100 \times .24}{\frac{660}{.60 \times \sqrt{2}} + \frac{5000}{4} + 1{,}080}$$

$$= 100.2 \text{ cu. ft. per sec.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

NOTE.—In the following examples, in order to avoid confusion, the values of f and v_1 used will always be the mean values for the class designated, as given in Tables IV and V. The value of v will be assumed as given in Table VI. Districts will be assumed to be rectangular; and, in each case, the outlet of the sewer will be considered to be at the lower edge of the district. The total discharge will generally be expressed to the nearest tenth of a cubic foot.

1. If, as in the example just given, the distances k, l, and d remain equal to 660, 5,000, and 280 feet, respectively, but the character of the district and surface is assumed to correspond to class f, Tables IV and V, while v is taken as in class d, Table VI, and S is taken as 10 feet per hundred, what will be the required capacity of the sewer at its outlet? Ans. 246.2 cu. ft. per sec.

2. For the same conditions as in example 1, what will be the flow per acre? Ans. 1.539 cu. ft. per sec.

3. What will be the discharge of the sewer considered in examples 1 and 2 at a point 2,640 feet above the outlet? Ans. 149.1 cu. ft. per sec.

4. If, for the same district, the character of the surface is taken to correspond to class c, v is taken as in class a, and S is taken as .25, what will be the maximum discharge at the outlet of the sewer? Ans. 33.9 cu. ft. per sec.

5. In a drainage district 4,000 feet long and 4,356 feet wide, the upper inlet to the sewer is 200 feet below the upper edge of the district. If f and v_1 are taken as in class e, v as in class b, and S is taken as 5 feet per hundred, what will be the maximum discharge at the outlet? Ans. 306.0 cu. ft. per sec.

OTHER FORMULAS FOR EFFLUENT

61. Buerkli's Formula.—Various other formulas have been proposed for the capacities of storm-water sewers. Of these, the formula proposed by Buerkli, a German authority, is probably the most reliable; it may be written as follows:

$$F = f_1 r_1 \sqrt{\frac{S_1}{A}}$$

in which F = flow of storm water per acre, in cubic feet per second;

S_1 = average surface slope (presumably toward and along the drain), in feet per *thousand feet* through drainage district;

A = area of drainage district, in acres;

f_1 = coefficient relating to "the proportion of rainfall that will reach the sewer";

r_1 = coefficient representing rate of rainfall, in inches per hour, "during period of greatest intensity of rain."

As S_1 is the average slope or fall of the surface in feet in 1,000 feet, it will always be equal to ten times the average slope expressed in feet in 100 feet; that is, $S_1 = 10\,S$. See Art. **55.**

62. Values of Coefficients in Buerkli's Formula. To the coefficient f_1 in the Buerkli formula are given values ranging from .31 in rural districts and suburbs to .75 in cities well built up, with a mean value of .62; for purposes of comparison, as applied to various classes of districts, assumed values of f_1 are given in the last column of Table IV. When not otherwise specified, a mean value of .625 will be used here. By *mean value* is here meant that value which best represents the most usual conditions.

The quantity r_1, though commonly stated as the rate of rainfall during the greatest downpour, has been shown to be scarcely more than an arbitrary coefficient. In climates where the intensity of rainfall varies greatly with the duration of the storm, it is necessary, in using r_1, to fix on a

definite length of time as representing the duration of a typical storm, and this is equivalent to arbitrarily fixing the value of r_1. When the length of a typical storm has been decided on, the value of r_1 will be the same as the value of y given by formula **1**, Art. **29**, or, generally, the same as given in Table II. In using the Buerkli formula, the European practice is to give r_1 values ranging from 1.75 to 2.5 inches per hour, but recent American practice gives r_1 values of from 2 to 3.5, and even higher, for sewers designed to carry all the storm water. In St. Louis, Missouri, a value of .75 for f_1 and values for r_1 varying from 3.02, for a district containing 100 acres, to 3.51, for a district containing 2,000 acres, were used. Observations taken in Rochester, New York, of rain storms lasting less than 1 hour indicate that, for the conditions in that city, storms lasting 51 minutes give the greatest flow. For storms of this duration, the value of r_1, taken equal to y as given by formula **1**, Art. **31**, will be 1.96, or, say, 2. A value of 2.75, which is about the mean of American practice, will be taken here for r_1.

63. The Total Effluent.—If both terms of the formula in Art. **61** are multiplied by A, the number of acres drained, and if in the first member E is written for $A\,F$, the value of the total effluent E will be as follows:

$$E = f_1\, r_1\, A \sqrt[4]{\frac{S_1}{A}} = f_1\, r_1 \sqrt[4]{S_1\, A^3} \qquad (1)$$

If both members of formula **1** are divided by $f_1\, r_1$, there results

$$\frac{E}{f_1\, r_1} = \sqrt[4]{S_1\, A^3} \qquad (2)$$

In Table VII, the values of the expression $\sqrt[4]{S_1\, A^3}$, which will be denoted by e, are tabulated for various slopes S_1 and areas A.

Replacing $\sqrt[4]{S_1\, A^3}$ by e, formula **2** may be written in the form

$$E = f_1\, r_1\, e \qquad (3)$$

in which the value of e is to be taken from Table VII, while the values of f_1 and r_1 will remain as before.

Buerkli fixes the greatest necessary capacity of storm-water sewers as .86 of a cubic foot per second per acre. This, however, is merely a general approximation. The required capacity may be greatly affected by the varying conditions relating to different cases, such as the size and shape of the district and the character and slope of its surface.

TABLE VII

VALUES OF e, OR $\sqrt[4]{S_1 A^3}$

Acres = A	$S_1 = 2.5$	$S_1 = 5$	$S_1 = 10$	$S_1 = 15$	$S_1 = 20$	$S_1 = 25$	$S_1 = 50$	$S_1 = 100$
40	20.00	23.78	28.28	31.30	33.64	35.57	42.29	50.30
60	27.10	32.24	38.34	42.43	45.59	48.21	57.33	68.17
80	33.64	40.00	47.57	52.64	56.57	59.81	71.13	84.59
100	39.76	47.29	56.23	62.23	66.87	70.71	84.09	100.00
120	45.59	54.22	64.47	71.35	76.67	81.07	96.41	114.65
160	56.57	67.27	80.00	88.53	95.14	100.60	119.63	142.26
200	66.87	79.53	94.57	104.66	112.47	118.92	141.42	168.18
300	90.64	107.79	128.19	141.86	152.44	161.19	191.68	227.95
400	112.47	133.74	159.05	176.02	189.15	200.00	237.84	282.84
500	132.96	158.09	188.02	208.09	223.61	236.44	281.17	334.37
600	152.44	181.28	215.58	238.58	256.37	271.08	322.37	383.37
800	189.15	224.92	267.50	296.03	318.11	336.36	400.00	475.68
1,000	223.61	265.90	316.23	349.96	376.06	397.64	472.87	562.34
1,200	256.37	304.84	362.57	401.24	431.17	455.90	542.16	644.74
1,500	303.08	360.39	428.62	474.34	509.71	538.96	640.93	762.20
2,000	376.06	447.21	531.83	588.57	632.46	668.74	795.27	945.74
2,500	444.57	528.68	628.72	695.79	747.67	790.57	940.15	1,118.03

face. The Buerkli formula is an attempt to embrace these conditions, and is one of the best formulas that have been proposed. It approximately involves most of the conditions materially affecting the effluent, and, when used intelligently, may be relied on to give reasonably accurate results. The formula, however, does not take the form of the district into account.

64. McMath Formula: St. Louis.—A formula similar in form to the Buerkli formula, except for the index of the

root, was derived from conditions in St. Louis, Missouri. For the flow per acre, it may be written thus:

$$F = f_1 r_1 \sqrt[4]{\frac{S_1}{A}} \qquad \textbf{(1)}$$

or, for the total effluent,

$$E = f_1 r_1 \sqrt[4]{S_1 A^3} \qquad \textbf{(2)}$$

in which all letters represent the same values as in the formula given in Art. **61**, and formula **1**, Art. **63**. As the formula was used in St. Louis, r_1 was taken equal to 2.75. In that city, sewers having a capacity less than that given by this formula, with values of .75, 2.75, and 15 used for f_1, r_1, and S_1, respectively, are known to be overtaxed. It is stated that, with the proper values substituted for the coefficients, the Buerkli formula will give results corresponding as well with the conditions observed in St. Louis as the McMath formula.

65. Board of Sanitary Engineers: Washington. The formula adopted by the Board of Sanitary Engineers, appointed to report on the sewerage of the District of Columbia (Art. **27**), is as follows:

$$E = 5.886\, A^{\frac{3}{4}} \qquad \textbf{(1)}$$

in which E = total effluent from district drained;
A = the number of acres in district.

This formula is equivalent to the Buerkli formula, with values of .75, 3.51, and 25 used for f_1, r_1, and S_1, respectively. It may, therefore, be readily used in connection with Table VII.

The recent practice in the city of Washington is to provide capacities for the storm-water sewers sufficient to carry the flow given by the following formulas:

For areas of 10 acres or less,

$$E = 3\,A \qquad \textbf{(2)}$$

For areas of from 10 to 60 acres,

$$E = 2\,A \qquad \textbf{(3)}$$

For areas of more than 60 acres,

$$E = 5.293\, A^{\frac{3}{4}} \qquad \textbf{(4)}$$

EXAMPLE.—For the example solved in Art. **60**, what will be the total flow or effluent as given by the Buerkli formula, assuming for r_1 a value of 2.50 and for f_1 and S_1 values corresponding to those assumed for f and S, respectively, in that example?

SOLUTION.—The number of acres in the district, as given by formula **1**, Art. **59**, is

$$\frac{660 \times (5,000 + 280)}{21,780} = 160$$

The slope S is 2 ft. in 100, and, consequently, the slope S_1, 1,000, is $10 \times 2 = 20$. From Table VII, the value of e, for a district containing 160 acres and having a surface slope of 20, is 95.14. As given in Table IV, the value of f_1 for class d is .44. Hence, by formula **3**, Art. **63**,

$E = .44 \times 2.50 \times 95.14 = 104.65$, or, say, 105 cu. ft. per sec Ans.

EXAMPLES FOR PRACTICE

NOTE.—The following examples relate to those given in Art. **60**. When not otherwise stated, they are to be solved by the Buerkli formula for the conditions given in those examples. The values of f_1 will be taken as given for the various classes of districts in Table IV; the value of S_1 will be taken as ten times the value stated for S, and, when not otherwise stated, the value of r_1 will be taken as 2.75.

1. What will be the total flow, in cubic feet per second, from the district described in example 1? Ans. 224.5 cu. ft.

2. What will be the required discharge for example 3? Ans. 145.4 cu. ft.

3. What will be the total effluent for the conditions stated in example 4, in cubic feet per second? Ans. 58.3 cu. ft.

4. What will be the total effluent from the same, in cubic feet per second, assuming the value of r_1 to be 1.75? Ans. 37.1 cu. ft.

5. For the district described in example 5, what will be the total flow per second, assuming a value of 2.50 for r_1? Ans. 309.2 cu. ft.

QUANTITY OF SEWAGE IN SEPARATE SYSTEM

66. General Considerations.—The design of the sewers of a separate system is based on the quantity of sewage delivered to each sewer or branch by the district tributary to it. The capacity of the system may also be affected by certain conditions of subsoil and ground water. An endeavor should be made so to construct the sewers that they shall be water-tight and allow no infiltration of ground water. It is difficult to do this, particularly where quicksand and water-bearing porous soils are found; experience shows that, even with careful oversight and reasonably good work, sewers laid in such soils often gather a considerable amount of ground water, and it is wise to make some allowance for this in determining the required capacity of the sewers. Cases have occurred where a lack of this precaution has resulted in the entire capacity of the sewers being taken up by ground water that percolated through the joints, leaving no capacity for sewage.

It is sometimes desirable to lay tile drains in the trenches with the sewers, for the purpose of permanently draining the subsoil. When this is done, it is usual to discharge the tile drains at a separate outlet, if convenient. If, however, no such outlet can be found, and the drains discharge into the sewers at manholes, provision should be made for this added flow in proportioning the sizes of the sewers. No rule can be given for determining the amount of subsoil water that may be added to the flow in either of these two ways. For 8-inch pipes, it runs from 5,000 gallons per mile per day to 25,000 gallons or more. It can only be determined approximately from previous experience in similar cases and from what knowledge can be secured as to the subsurface conditions.

67. Sewage Discharge and Water Supply.—The available records of sewer gaugings for American cities are not sufficient to indicate accurately the quantity of sewage per capita that must be provided for. Records of water supply, however, are abundant, and, since the sewer gaugings that have been made indicate that the quantity of sewage from a given district is somewhat less than the quantity of water consumed by its inhabitants, the statistics of water supply are useful and are the main factor in estimating the sewage discharge.

In using the records of a public water supply for this purpose, it must be remembered that often there are factories that have a private water supply, and these may often discharge a considerable volume of sewage, which should be provided for. The provision necessary for subsoil water has already been referred to. That the amount of actual sewage will generally be less than the water supply will be evident when it is considered that all the water used for sprinkling, and some of that used for cleaning, either soaks into the ground or evaporates. In manufacturing districts, also, considerable quantities of water are used that do not reach the sewers.

68. Average Rate of Water Consumption.—In *Water Supply*, Part 1, may be found a discussion of the variation of water consumption for different times. It is there shown that the average rate of water consumption in American cities having a population varying from 10,000 to 100,000 is probably not much below 100 gallons per capita a day. The average for large cities is generally somewhat greater than for small cities. Cities that are well supplied with water meters have an average as low as 30 gallons, and some of the larger cities with abundant water supply use 200 gallons or more. European cities use about one-half or two-thirds as much as American cities.

69. Variations in the Water Consumption and Sewage Flow.—The foregoing statements relate to the average daily water consumption and the corresponding sewage discharge. It is important to note, however, that

the consumption of water, and, consequently, the discharge of sewage vary greatly.

In the design of sewerage systems, two periods of fluctuation must be considered; namely, the daily fluctuations during the year, giving what is called the *day maximum*, and the hourly fluctuations during the day, giving what is known as the *hour maximum*. The hourly fluctuations generally occur with a considerable degree of regularity, while the daily fluctuations are less regular. There are also well-defined weekly fluctuations, the consumption being generally greater on Monday than on other days of the week. The hourly fluctuations in the water consumption and sewage discharge generally occur at rather regular periods during the 24 hours. The maximum rate is generally reached an hour or so before noon, and may attain a rate one and one-half times the mean hourly rate, and in extreme cases even a much higher rate.

There are generally two periods of excessive water consumption during the year, either of which may be the maximum. One period occurs during the coldest weather, and the other during the warm, dry period that generally comes in late summer. The maximum of the cold weather is the one that principally affects the sewage discharge. In severe weather, faucets are left running to prevent freezing, thus making considerable waste. These and other conditions present during cold weather will, in most cases, produce the maximum rate of water consumption during the year. Nearly the entire volume of this cold-weather maximum will enter the sewer. During the maximum of the warm and dry weather, much water will be used for sprinkling and similar purposes; this water, being absorbed by the ground and atmosphere, will not reach the sewer. The daily average for the maximum day may be 50 per cent., and in extreme cases more than 75 per cent., higher than the mean daily rate for the year.

70. Quantity of Sewage as Determined by Actual Measurement.—Probably no *continuous* record of sewage flow that would be serviceable in proportioning the size of sewers of the separate system can be found. Records have

been kept for limited periods, but in many cases the sewage has been diluted with ground water to a degree that could not well be determined, and the records have but a limited usefulness as a basis for sewer design.

In Fig. 5, taken from "Engineering News," is a graphic record of sewer gaugings that were taken for 24 hours at about 10-minute intervals. These observations were made on February 7 and 8, 1902, in fairly cold weather and in a northern American city. The soil in which the sewers lie is a dense clay, and probably no subsoil water was mixed with

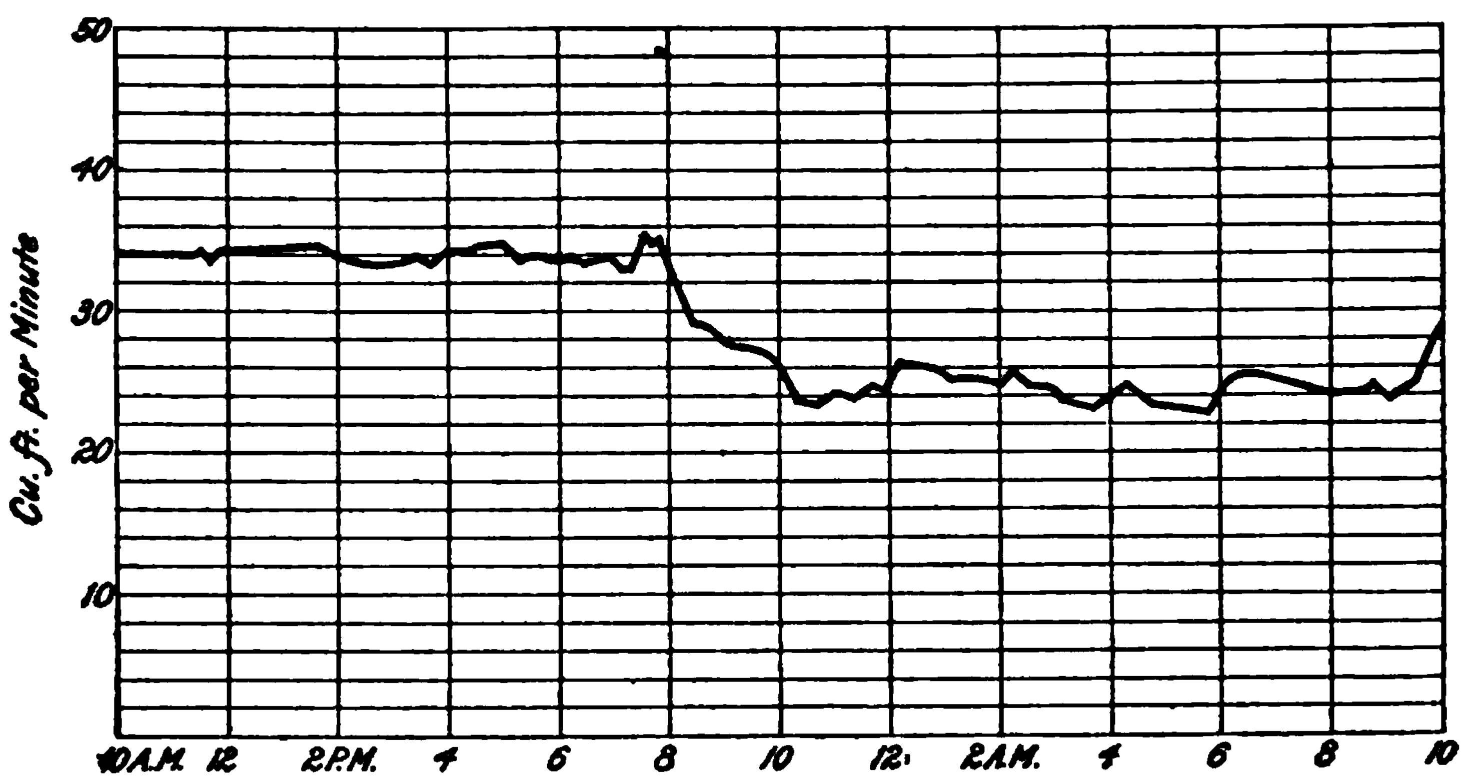


FIG. 5

the sewage. The diagram fairly represents the variations in sewage discharge for that time and place, and may be taken as a typical cold-weather diagram, where water is allowed to run at night to prevent freezing.

The following is a summary of the results:

Maximum rate	35.28 cubic feet per minute
Average rate	28.78 cubic feet per minute
Minimum rate	22.15 cubic feet per minute

Taking the average rate as 100, the rates were as follows:

Maximum rate	125
Average rate	100
Minimum rate	77

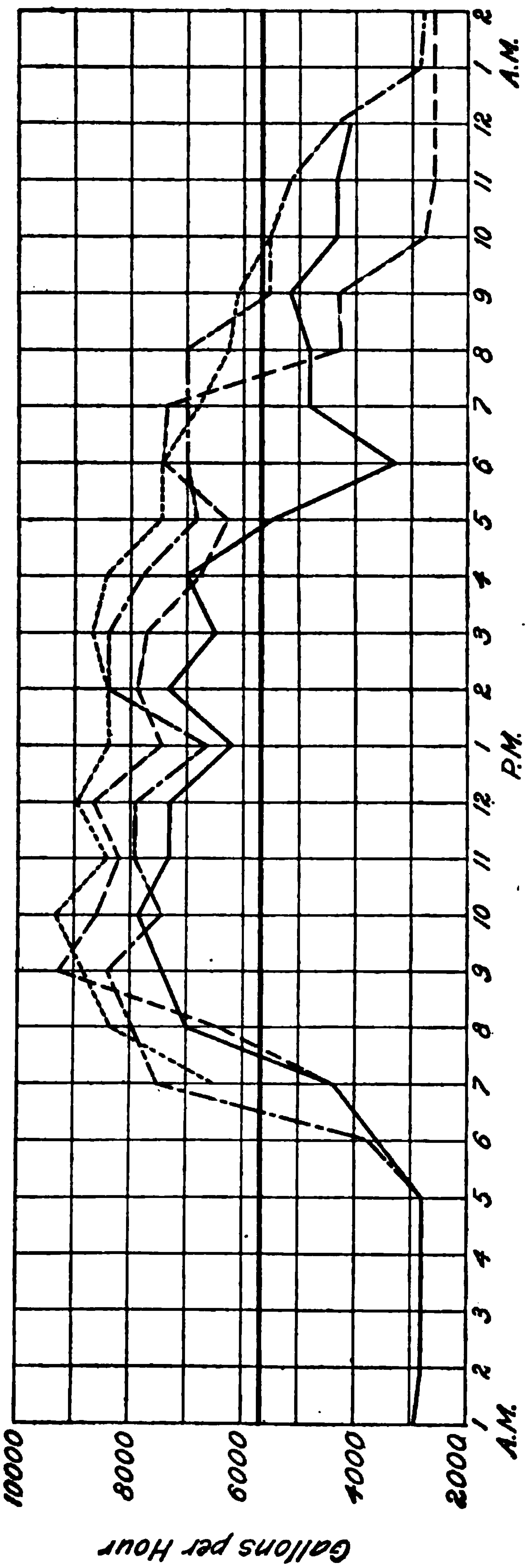

10000
8000
6000
4000
2000
Gallons per Hour
1 2 3 4 5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8 9 10 11 12 1 2
A.M.
P.M.
A.M.

FIG. 6

Fig. 6, from Ogden's "Sewer Design," shows a similar record for four successive days, made at Chautauqua, New York, in the summer of 1893. The diagram is fairly typical of ordinary conditions. The heavy horizontal line shows the average flow.

The following is a summary of the results:

Maximum rate	20.5 cubic feet per minute
Average rate	12.7 cubic feet per minute
Minimum rate	6.2 cubic feet per minute

Taking the average rate as 100, the rates were:

Maximum rate	162
Average rate	100
Minimum rate	49

71. The common practice among American engineers is to proportion the sewers of the separate system so that, when running half full, they will discharge a quantity of sewage equal to the maximum hourly water consumption, this maximum being taken equal to 1.5 times the average. The remaining capacity is reserved for extreme variations in flow and for the passage of air for ventilation. The conditions of flow are then as follows:

Average daily flow, 100 per cent.; sewer one-third full.

Average maximum daily flow, 150 per cent.; sewer one-half full.

Total capacity of sewer, 300 per cent.; sewer full.

The average daily flow is assumed to be such as may reasonably be expected when the territory is fairly well developed and the buildings all connected with the sewers.

EXAMPLE.—What capacity should the main sewer of a city of 25,000 population have, the water consumption being 85 gallons per head each day, assuming the sewer to be flowing half full?

SOLUTION.—The total water consumption is $25{,}000 \times 85 = 2{,}125{,}000$ gal. per day. The discharge from the sewer is $2{,}125{,}000 \times 1.50 = 3{,}187{,}500$ gal. per day. Reducing this quantity to cubic feet per second, the capacity of the sewer is found to be

$$\frac{3{,}187{,}500}{7.48 \times 24 \times 60 \times 60} = 4.9 \text{ cu. ft. per sec.} \quad \text{Ans}$$

SEWERAGE

(PART 2)

ARRANGEMENT OF PLANS FOR SEWERAGE SYSTEMS

1. General Considerations.—The sewers of a town should be laid out in such a manner and according to such a system as will best conform to the topography of the surface, and fulfil the requirements for efficient sewerage. In most cases, the grades of the sewers should, in a general way, conform to the slope of the surface. In dividing the territory of a city into sewer districts, and laying out the main sewers, no definite plan can be rigidly followed. Certain methods of dividing the territory and laying out the systems of sewers, however, may be recognized by their general and characteristic features. The five more prominent of these will now be noticed. They will here be designated as the *perpendicular*, *intercepting*, *fan*, *zone*, and *radial* plans.

2. Perpendicular Plan.—Where a city is bounded on one side by a body of water, or is divided by a stream of water flowing through it, the area is generally laid out into a number of districts having entirely distinct systems. Each district has its trunk sewer, which has a direction approximately perpendicular to the body of water into which it discharges, and, generally, a number of branch sewers discharging into the trunk sewer.

This plan is shown in Fig. 1. It is assumed that the general surface of the town slopes rapidly toward the river and

Fig. 1

also gently in the direction of the current of the river, as shown by the contour lines. The town is shown completely sewered by means of trunk sewers running down nearly every street leading to the river. The districts tributary to some of the trunk sewers are of considerable extent, while those tributary to others are small. The course of the sewer is, in each case, governed by the inclination of the surface and by other local conditions. Although towns are not generally sewered as closely and completely as here shown, the figure will serve as an illustration of a fairly complete perpendicular system.

This plan has the advantage of giving sewers of the shortest length and smallest section possible to any locality to which it is adaptable. It is generally not only the cheapest but also the most convenient plan, and is the one usually adopted by a town before any complete system is designed. The principal disadvantage of the plan is the pollution, within the limits of the town, of the stream or other body of water into which the sewage is discharged.

3. Intercepting Plan.—The intercepting plan is similar to the perpendicular plan, except that large sewers, called **intercepting sewers,** are constructed along the banks of the river or body of water to intercept the sewage discharged by the sewers of the perpendicular plan and convey it to an outlet below the city, or to suitable points for treatment. This removes the disadvantages of the perpendicular plan. While the intercepting plan is not generally the best plan possible for a given district where the entire system can be designed and constructed new, it is usually the best plan to adopt where portions of the system have been previously constructed, or as a modification of the perpendicular plan. The perpendicular plan shown in Fig. 1 is shown in Fig. 2 modified to the intercepting plan.

4. Fan Plan.—In some cases, the sewage from an entire city may be conveyed to a single outlet by means of a number of converging trunk sewers and their various branches. The form of the entire system will somewhat approximate

Fig. 2

FIG. 8

the form of a fan, or the skeleton of a leaf. This plan is especially adaptable to a city having a surface contour somewhat of the general form of a basin. In such a case, the district comprising the center of the city is usually much larger than the others. The plan is also adaptable to other forms of surface.

The fan plan is, generally, a somewhat more direct system than the intercepting plan, and leads to practically the same results. It is the plan best adapted to average conditions.

In Fig. 3, the fan plan is shown applied to the same town to which the perpendicular and intercepting plans are shown applied in Figs. 1 and 2. In this plan, however, the general slope of the surface is assumed to take the form of a half basin sloping from every direction toward the point *a*, as shown by the contour lines.

5. Zone Plan.—The **zone plan** is adaptable to a city in which the surface consists of a series of plateaus; these generally rise in successive steps, the highest being the one most remote from the watercourse. Each plateau has its own distinct system, which may be either on the fan or on the intercepting plan. The different systems may be connected for flushing purposes. A very great adyantage of this system is that it diverts the water and sewage of the upper plateaus away from the lower portions of the city, thereby obviating the danger of flooding. In Fig. 4, this plan is shown applied to the same town to which the other plans have been applied in the three preceding figures. The general form of the surface assumed in this case is shown by the contour lines.

6. Radial Plan.—In the **radial plan**, the city is divided into a number of sectors, corresponding somewhat to sectors of a circle, and the sewage from each sector is carried from the center outwards. The position of the trunk sewer in each sector is approximately that of a radial line. This plan is adaptable to a city whose greatest elevation is in the central part; for a iarge city having such form of surface, it is generally the best plan.

136—18

One great advantage of this plan is that the sewers are all small in the center of the city, and become larger only as their distances from the center become greater. Moreover, the sewers in the center of the city will generally be of as great capacity as will be required for a very long time. As the city grows, it will generally be sufficient to extend the sewers outwards with enlarged sections. The old sewers will generally be of sufficient capacity, as there will be no important extensions of branch sewers above.

In the plans described in previous articles, the trunk sewers either must be designed with capacity sufficient for future requirements, which it is very difficult to estimate accurately, or must be from time to time enlarged to provide for the sewage from new lines of tributary branches, which must be added as the city grows and new districts are annexed. If a trunk sewer is designed with a capacity sufficient for the estimated future requirements, it will be much more expensive than required for present purposes; and, moreover, the growth of the city may be such as to exceed the estimated requirements and necessitate the construction of new sewers. These new sewers, being trunk sewers, and being generally in or near the center of the city, will be expensive in their construction, for each of the plans now under consideration involves the conveying of more or less sewage from the outskirts directly through the center of the city. The limits of a city may be extended indefinitely, and the volume of sewage from the outskirts correspondingly increased.

These difficulties are obviated in the radial plan, which is especially advantageous for large cities. The chief objection to this plan is the difficulty in obtaining suitable outfalls for the sewers; this often makes pumping necessary.

7. Remarks on the Different Plans.—In very many cases, the requirements will best be fulfilled by a combination of two or more of the foregoing plans. In general, it may be stated that the greatest degree of economy is obtained

by concentrating the systems as much as possible. A few reasonably large sewers can be constructed more cheaply than a large number of small sewers having the same capacity.

It is, therefore, probable that, if all considerations of future requirements are neglected, the fan plan is, for the greater number of cases, the most economical. The intercepting plan can generally be made to serve most economically the same purpose as the fan plan in cities that are partly sewered. For cities in which the surface slopes outwards from the center toward lower points that afford suitable outlets, the radial plan is usually the best.

SEWER COMPUTATIONS

CIRCULAR SEWERS

8. Fundamental Formulas.—The fundamental formulas used in sewer design are (see *Hydraulics*)

$$v = c\sqrt{rs} \qquad (1)$$

$$Q = Fv \qquad (2)$$

in which v = velocity of flow, in feet per second;
r = hydraulic radius of sewer, in feet;
s = sine of inclination, or rate of grade;
Q = discharge of sewer, in cubic feet per second;
F = area, in square feet, of cross-section of the flowing water or sewage;
c = a coefficient, usually taken as given by Kutter's formula.

In sewers, the minimum value of v is determined by that velocity which is so low that material in suspension will settle and form deposits in the sewer. This minimum velocity is taken as about 2 feet per second for ordinary sizes, and a sewer should be so designed that the velocity will not be less than this. The maximum velocity is determined by the fact

that velocities above about 10 feet per second will gradually wear away brickwork, so that the sewers may be destroyed by erosion. Between these limits, the velocity will depend, by formula **1**, on the slope and hydraulic radius. Since separate sewers are always designed to flow half full, and for that depth r is equal to one-fourth the diameter d, $\frac{d}{4}$ may be substituted for r in formula **1**.

Storm sewers are designed to flow full, but r has the same value, $\frac{d}{4}$, as for a half-full sewer.

9. Value of Coefficient in Kutter's Formula.—The value of c to be used in Kutter's formula is given by the equation (see *Hydraulics*, Part 3)

$$c = \frac{23 + \frac{1}{n} + \frac{.00155}{s}}{.5521 + \left(23 + \frac{.00155}{s}\right)\frac{n}{\sqrt{r}}}$$

For sewer work, two values of n are used: .013 for vitrified pipe, and .015 for concrete and brick sewers.

Values of c corresponding to different slopes and to different values of n are tabulated in some works, among them being Trautwine's "Civil Engineer's Pocket Book." Such tables are very valuable, as they save a great deal of time. Intermediate values can be found by interpolation.

10. Application of Formula.—The sewer problems to be solved by means of formulas usually appear in one of the following forms, two factors out of the four Q, d, s, and v being always given:

1. Given the quantity Q and the grade s, to determine the diameter d and the velocity v.
2. Given the quantity Q and the diameter d, to determine the slope and the velocity.
3. Given the quantity Q and the minimum or maximum velocity, to determine the diameter and the grade.
4. Given the diameter d and the grade s, to determine the velocity v and the quantity Q.

EXAMPLE 1.—If the discharge from a pipe sewer flowing half full is 4 cubic feet per second and the grade is .0016, what are the values of the diameter d and the velocity v?

SOLUTION.—From formula **2**, Art. **8**,

$$Q = Fv = \tfrac{1}{2} \times \frac{\pi}{4} \times d^2 v;$$

whence $$v = \frac{8\,Q}{\pi\, d^2}$$

Substituting this value of v in formula **1**, Art. **8**,

$$\frac{8\,Q}{\pi\, d^2} = c\sqrt{r\,s}$$

Squaring, writing $\frac{d}{4}$ for r, and solving for d,

$$d = \sqrt[5]{\frac{256\,Q^2}{\pi^2\, s\, c^2}}$$

Now, substituting the given value of s and that of $n = .013$ in the formula of Art. **9**, we have

$$c = \frac{23 + \dfrac{1}{.013} + \dfrac{.00155}{.0016}}{.5521 + \left(23 + \dfrac{.00155}{.0016}\right) \times \dfrac{.013}{\sqrt{\dfrac{d}{4}}}} = \frac{100.97}{.5521 + \dfrac{.62322}{\sqrt{d}}}$$

Writing this value for c in the foregoing expression for d, and substituting the given values of Q and s,

$$d = \sqrt[5]{\frac{256 \times 4^2\left(.5521 + \dfrac{.62322}{\sqrt{d}}\right)^2}{\pi^2 \times .0016 \times 100.97^2}} = 1.910\sqrt[5]{\left(.5521 + \frac{.62322}{\sqrt{d}}\right)^2} \qquad (1)$$

From this equation, the value of d is found by successive trials. A value is first assumed for d, and substituted in the last member of the equation. If the result is not sufficiently close to the assumed value, that result, which will generally be a close value of d, is treated as a new assumed value, and substituted in the last member of the equation; and so on, until a close agreement is found. Here, a value of 21 in., for 1.75 ft., will first be assumed or d. Equation (1) then becomes, using only four significant figures,

$$d = 1.910\sqrt[5]{\left(.5521 + \frac{.6232}{\sqrt{1.75}}\right)^2} = 1.93 \text{ ft., nearly}$$

As this value is not sufficiently close to the one assumed, another trial will be made. Writing 1.93 for d in the last member of equation (1), and using only three significant figures,

$$d = 1.91\sqrt[5]{\left(.552 + \frac{.623}{\sqrt{1.93}}\right)^2} = 1.911 \text{ ft.} = 23 \text{ in., nearly}$$

As this result agrees very closely with the value assumed (1.93), it may be adopted; but, the nearest commercial size being 24 in., the latter size will be used. Substituting in formula **1**, Art. **8**, this value (that is, 2 ft.) and the value of c just given, there results

$$v = \frac{100.97}{.5521 + \frac{.62322}{\sqrt{2}}} \times \sqrt{.5 \times .0016} = 2.9 \text{ ft. per sec.} \quad \text{Ans.}$$

EXAMPLE 2.—The diameter of a pipe sewer is 30 inches, and the grade .001. Required the velocity v and the discharge Q, when the sewer is flowing half full.

SOLUTION.—Substituting the values $n = .013$, $s = .001$, and $r = \frac{d}{4} = \frac{30}{12 \times 4} = .625$ in the formula of Art. **9**,

$$c = \frac{23 + \frac{1}{.013} + \frac{.00155}{.001}}{.5521 + \left(23 + \frac{.00155}{.001}\right) \times \frac{.013}{\sqrt{.625}}} = 106.2$$

Substituting this value of c and the given values of r and s in formula **1**, Art. **8**,

$$v = 106.2\sqrt{.625 \times .001} = 2.7 \text{ ft. per sec., nearly.} \quad \text{Ans.}$$

From formula **2**, Art. **8**,

$$Q = \frac{\pi d^2}{8} v$$

Substituting known values in this formula,

$$Q = \frac{3.1416 \times 2.5^2}{8} \times 2.7 = 6.6 \text{ cu. ft. per sec.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. What is: (*a*) the velocity, and (*b*) the discharge of an 8-inch pipe sewer flowing half full on a grade of .4 foot in 100?

Ans. { (*a*) 2 ft. per sec.
(*b*) .35 cu. ft. per sec.

2. What is the required diameter of a pipe sewer flowing full to carry off a flow of 3.6 cubic feet per second, the grade being .125 foot in 100? Ans. 18 in.

3. What will be the velocity in a 20-inch pipe sewer flowing full, when the discharge is 8.5 cubic feet per second? Ans. 3.9 ft. per sec.

4. What will be the discharge in a pipe sewer 24 inches in diameter flowing half full, when the velocity is 2.5 feet per second?

Ans. 3.9 cu. ft. per sec.

11. Use of Diagrams.—In order to facilitate computations, diagrams are often used from which the required quantities can be easily read off. Such diagrams are prepared in a great variety of forms. Fig. 5 shows a form prepared by G. S. Pierson. The grades per cent. ($100s$) are laid off, to any convenient scale, on a horizontal line OX; discharges, in cubic feet per second, are laid off on a vertical line OY; and horizontal lines are drawn through the points of division *1*, *2*, *3*, etc. If desired, the spaces *1–2*, *2–3*, etc. may be divided into halves, quarters, or tenths. In the figure, there are two auxiliary vertical lines O_1Y_1 and O_2Y_2, where the discharges are laid off in gallons per minute and gallons per day, respectively. Assuming any diameter, as 6 inches, the discharges for the different grades, *.1*, *.2*, *.3*, etc. are computed by Kutter's formula, and laid off as ordinates from the points *.1*, *.2*, etc., on OX. Thus, the ordinate $3\text{–}a_1$ represents the discharge corresponding to a grade of 3 per cent. and a diameter of 12 inches; the ordinate $5\text{–}a'$ represents the discharge corresponding to the same diameter and a grade of 5 per cent. In a similar manner, other points are found for the same diameter; joining all these points by a line (which is not exactly straight), the 12-inch diameter line, starting from O, is obtained. Other diameter lines are drawn in the same general way.

The curved velocity lines are drawn as illustrated by the following example: Having laid off the ordinates $3\text{–}a_1$ and $3\text{–}a_2$, for a 3-per-cent. grade and diameters of 12 and 15 inches, respectively, the corresponding velocities are computed, and written near the points a_1 and a_2. Suppose those velocities to be, respectively, 7.4 and 8.8 (these figures are here assumed as rough approximations for the purposes of illustration). To determine a point p corresponding to a velocity of 8, the distance a_1a_2 is divided into two parts proportional to $(8 - 7.4)$ and $(8.8 - 8)$, or to 6 and 8, making $a_1p = \frac{6}{14}\,a_1a_2$, $a_2p = \frac{8}{14}\,a_1a_2$. Having found other velocity points in a similar manner, a curve is drawn through each series of points corresponding to the same velocity. It should be

observed that, in connection with this diagram, the velocities need not be very accurate.

The use of the diagram is explained in the following examples.

EXAMPLE 1.—Given a discharge of 9.2 cubic feet per second, and a grade of .042 ($= s$), required the necessary diameter of sewer.

SOLUTION.—A horizontal line may be imagined drawn through a point on OY at a distance from *9* equal to .2 of the distance *9–10*. The grade per cent. is $100s = 100 \times .042 = 4.2$. Taking the abscissa 4.2 on OX, and following down the corresponding ordinate, to its intersection (estimated by eye) with the horizontal line mentioned, this intersection is seen to have about the position a. As this point lies between the 12-in. and the 15-in. lines for diameters, a 15-in. sewer will be used. Ans.

EXAMPLE 2.—The diameter of a sewer being 18 inches, and the discharge 11.2 cubic feet per second, to determine the grade.

SOLUTION.—By referring to the diagram, Fig. 5, it is seen that the diameter line for 18 in. intersects on the ordinate *1.2* an imaginary horizontal line drawn at a distance 11.2 below OX. Therefore, the grade is 1.2 per cent. Ans.

EXAMPLES FOR PRACTICE

NOTE.—The following examples are to be solved by the diagram.

1. What should be the diameter of a sewer, if the discharge is 12.5 cubic feet per second and the grade is 1.8 per cent.? Ans. 18 in.

2. What should be the grade of a sewer 12 inches in diameter to discharge 5 cubic feet per second? Ans. 2.25 per cent.

3. What is the velocity in a sewer 18 inches in diameter laid on a grade of 1.5 per cent.? Ans. 7 ft. per sec.

4. What should be the grade of a 12-inch sewer to maintain a velocity of 4 feet per second? Ans. .9 per cent.

---

12. Sewer Tables.—Tables I, II, and III, at the end of this Section, give the velocities and discharges for sewers of different sizes laid on different grades. Table I is for vitrified pipe, with a value of $n = .013$; Table II is for brick circular sewers, with a value of $n = .015$; and Table III is for brick egg-shaped sewers, with a value of $n = .015$.

Fig. 5

(The subject of egg-shaped sewers will be treated at length in subsequent articles.) It will be noticed that the columns are not all full. For instance, in Table III, no discharge is given for a 66″ × 99″ egg-shaped sewer and a grade of .0070. The reason is that a sewer of this size would not be used with the given grade, as the velocity would be too great (see Art. 8).

To illustrate the use of the tables, suppose it is required to find the discharge of a 5-foot brick sewer flowing full on a .002 grade. Looking in Table II for 60 in the first column, and noticing that .002 falls between .001 and .0025, the discharge is found by interpolation to be 100.9 cubic feet per second. In the same manner, the velocity is found to be about 5 feet per second.

EXAMPLES FOR PRACTICE

NOTE.—The following examples are to be solved by means of the tables.

1. What quantity of sewage will a 4-foot brick sewer discharge on a .3-per-cent. grade? Ans 67.8 cu. ft. per sec.

2. What size egg-shaped sewer will discharge 85 cubic feet per second on a .2-per-cent. grade? Ans. 48 in. × 72 in.

3. On what grade should a 12-inch pipe sewer be laid to discharge 2.5 cubic feet per second flowing half full? Ans. 2.5 per cent.

4. What velocity would be found in an 8-inch pipe laid on a grade of .4 per cent.? Ans. 2 ft. per sec., nearly

13. Necessary Refinement.—It must not be inferred that the discharging capacity of a sewer can be foretold to a nicety. The computed capacity may not be within 10 per cent. of the actual capacity, owing to the uncertainty in the number and effect of Y branches, manholes, the manner of laying, and other factors, the effects of which cannot be exactly computed.

In pipe sewers, it is not practical to provide the exact computed capacity, for the reason that sewer pipes are manufactured only in certain definite sizes, those ordinarily used being 4, 6, 8, 9, 10, 12, 15, 18, etc. inches in diameter. Brick sewers are also generally built in diameters of 24, 27, 30, 33, 36, etc. inches. It is customary to adopt the next

size larger than that obtained by computation; so that extreme refinement in the computations is not necessary. Furthermore, there is to be taken into consideration the probable growth of the territory, and this introduces a factor depending on sound judgment rather than on over refinement in mathematical work.

The novice often falls into the error of assuming that, because the tables or diagrams give greater velocities for large sewers than for small ones laid at the same grade, the velocities will be increased by laying a large sewer, the grade being the same. This is not so. There can be only a definite quantity of sewage flowing, and if this is spread over the invert of too large a sewer, the velocity will be less and the danger of deposits greater.

14. Effect of Depth of Flow on Velocity.—The velocity will diminish very rapidly as the volume of sewage, and consequently the depth of the stream, becomes less. This is shown by the diagram of Fig. 6. In this diagram, the three curves represent, respectively, the relative values of the area of wetted cross-section, of the velocity, and of the discharge for different depths of flow, the value for a full depth of flow being taken as unity in each case. For any depth of flow, the value given by the area curve multiplied by the value given by the velocity curve will equal the value given by the discharge curve.

The relative values given by the curves of the diagram are for a uniform slope—that is, for any slope, provided they all relate to the same slope. The curves of velocity and discharge are, necessarily, only approximately exact, but they are sufficiently accurate for all ordinary computations. All the curves of the diagram relate to circular sewers.

The preceding formulas give the velocity and discharge for sewers flowing full. Sewers rarely discharge at their full capacity, and in practice should be designed so that they will not do so. The velocities that are shown on the diagram in Fig. 5 will, therefore, not be the true velocities, but will bear a relation to the true velocities that can be

ascertained (in circular sewers) from Fig. 6, according to the depth of flow.

As already stated, pipes are usually designed for a flow at half depth, at which depth the area and discharge are one-

Depth of Flow.

Comparative Area, Velocity or Discharge

FIG. 6

half the area and discharge of the full pipe, and the velocity is the same as for a full pipe.

EXAMPLE.—If the velocity in a sewer flowing full is 5.6 feet per second, what is the velocity when the sewer is flowing .8 full?

SOLUTION.—Referring to the diagram of Fig. 6, the abscissa of the velocity curve corresponding to the ordinate .8 is 1.16. The velocity is, then, $5.6 \times 1.16 = 6.5$ ft. per sec. Ans.

EXAMPLES FOR PRACTICE

1. If the discharge of a circular sewer flowing full is 12 cubic feet per second, what is the discharge: (*a*) when .9 full? (*b*) when .75 full? (*c*) when .3 full?

Ans. { (*a*) 13 cu. ft. per sec.
(*b*) 10.9 cu. ft. per sec.
(*c*) 2.3 cu. ft. per sec.

2. **If the area of a circular sewer is 4.82 square feet, what is the area of the water section whose depth is: (*a*) .8 of the diameter? (*b*) .5 of the diameter? (*c*) .1 of the diameter?**

Ans. (*a*) 4.15 sq. ft. (*b*) 2.41 sq. ft. (*c*) .24 sq. ft.

3. **If the velocity in a sewer flowing full is 4.5 feet per second, what is the velocity: (*a*) when .5 full? (*b*) when .7 full? (*c*) when .15 full?**

Ans. (*a*) 4.5 ft. per sec. (*b*) 5.1 ft. per sec. (*c*) 2.1 ft. per sec.

---

15. Concluding Remarks.—The final and most important question to be decided in designing a system of sewers is the question of size. This question can be decided much more definitely for sewers that are to convey only sewage proper than for those that are to convey storm water also, since the quantity of sewage can be more definitely predicted.

The capacity of the sewer must be sufficient to provide for the maximum rate of discharge. This has already been studied under that head in a general way, but should be investigated with particular reference to the case in hand. For sewers of the separate system, the required capacity may be materially greater in a manufacturing than in a residence district. The daily quantity of sewage from a manufacturing district may be discharged during a few hours, while that from the residence district will generally be distributed through the greater part of the 24 hours.

The waste of water is generally more nearly constant than its legitimate use. Consequently, the greater the percentage of water wasted, the more nearly uniform will be the discharge of sewage and the less will the maximum exceed the average discharge. These and all similar conditions must be carefully considered in all parts of the area to be sewered, and the required capacities of the sewers must be determined accordingly.

In order to maintain a cleansing velocity in the upper part of a sewer, where the depth of flow is only a small fraction of the diameter, the inclination of the sewer must be very much increased at these upper levels. It is generally

impractical to increase the grade near the dead ends of sewers enough to compensate for the decreased flow, and preserve a uniform velocity. It is for this reason that flush tanks have been adopted, which periodically increase the depth of flow and cleanse the sewer.

EGG-SHAPED SEWERS

16. Variation of Flow in Storm-Water Sewers.—The flow in a storm-water sewer will necessarily fluctuate greatly. During a violent storm, the sewer may be taxed to its full capacity; while, during an extended drouth, the flow may become very small. It is evident, however, that the sewer must be large enough to have sufficient capacity to carry its greatest flow. When the flow in a large circular sewer becomes very small, the cross-section of the flow will necessarily be very shallow, and, consequently, will have a very small hydraulic mean radius. It follows that, as the velocity varies approximately as the square root of the hydraulic mean radius, the velocity also will be small and in many cases insufficient to prevent the deposition of solid matter carried in the sewage. Hence, it will readily be discerned that the circular form of section is not well adapted to sewers in which the flow of sewage varies considerably.

17. Other Forms of Cross-Section.—As a result of the conditions just mentioned, other forms of cross-section, having greater values of the hydraulic mean radius for comparatively small cross-sections of flow, have been devised for large sewers. A variety of forms have been employed, especially in Europe. The form of cross-section, however, that most satisfactorily accomplishes this purpose, is the form known as **egg-shaped**. In this form, the radius of the upper part is greater than that of the lower part, so that the cross-section has somewhat the shape of an egg.

18. General Form of Egg-Shaped Sewers.—The general form of the cross-section of an egg-shaped sewer is shown in Fig. 7. The part above the line OO' is a semicircle; the part below the line OO' is formed by the three

arcs DE, EG, and GA, the arcs DE and GA having equal radii. It will be noticed that three different radii are used in constructing the figure; namely, $CA = CB = CD$ for the

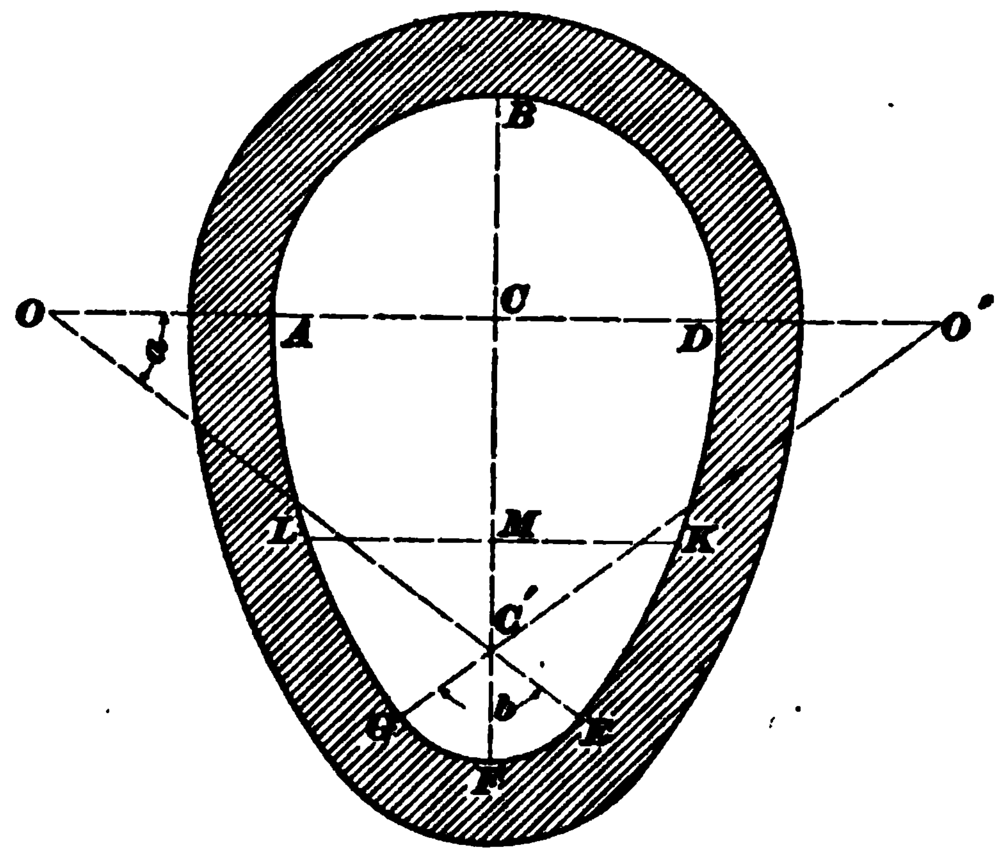


FIG. 7

upper semicircle, $OD = OE = O'G = O'A$ for the two side arcs DE and GA, and $C'E = C'F = C'G$ for the lower arc EFG, commonly called the **invert.**

19. General Formulas for Egg-Shaped Sewers. Referring to Fig. 7, let

$r = CA = CB = CD$ = radius of upper semicircle;
$r_1 = C'E = C'F = C'G$ = radius of lower arc;
$r_2 = OD = OE = O'G = O'A$ = radius of sides;
$d_h = AD$ = horizontal diameter;
$d_v = BF$ = vertical diameter;
$c = CC'$ = distance between centers;
a = angle COC' or $CO'C'$, subtended by side arc DE or GA;
b = angle $EC'G$, subtended by lower arc EFG;
P = inner perimeter $ABDEFG$;
A = area of internal cross-section;
R = hydraulic mean radius.

If the value of r is assumed, then,

$$d_h = 2r \qquad (1)$$

Writing the values of d_v and r_1 in terms of r,

$$d_v = r + r_1 + c;$$

whence $$c = d_v - (r + r_1) \qquad (2)$$

Also, $$(r_0 - r)^2 + c^2 = (r_0 - r_1)^2;$$

whence $$r_0 = \tfrac{1}{2}\left(\frac{c^2}{r - r_1} + r + r_1\right) \qquad (3)$$

$$\sin a = \frac{c}{r_0 - r_1} \qquad (4)$$

$$b = 180° - 2a \qquad (5)$$

By adding together the expressions for the values of the arcs ABD, DE, EFG, and GA, the following expression is obtained:

$$P = \pi\left[\frac{a}{90°}(r_0 - r_1) + r + r_1\right] \qquad (6)$$

Also, by adding together the expressions for the areas $ABDA$, $ACC'GA$, $CDEC'C$, and $CEFGC'$, the following expression is obtained:

$$A = \frac{\pi}{2}\left[r^2 + r_1^2 + \frac{a}{90°}(r_0^2 - r_1^2)\right] - c(r_0 - r) \qquad (7)$$

For sewer running full,

$$R = \frac{A}{P} \qquad (8)$$

20. Horizontal and Vertical Diameters.—The ratio between the horizontal and vertical diameters (d_h and d_v) that is most generally used is that of 2 to 3. In other words, $d_h = \frac{2}{3} d_v$ or, from formula **1**, Art. **19**,

$$d_v = 1\tfrac{1}{2} d_h = 3r \qquad (1)$$

Hence, from formula **2**, Art. **19**,

$$c = 2r - r_1 \qquad (2)$$

21. Cross-Sections of Flow.—The computations for the velocity and discharge of egg-shaped sewers are usually made for sewers running *full*, *two-thirds full*, and *one-third full*. By two-thirds and one-third full are meant a depth of flow equal to $\frac{2}{3} d_v$ and $\frac{1}{3} d_v$, respectively.

For the cross-section of a sewer in which the relations between d_h and d_v are as given by formula **1**, Art. **20**, if the semi-circumference ABD, Fig. 7, is subtracted from the perimeter P, the remainder will be the wetted perimeter for

a sewer flowing two-thirds full. Hence, for a depth of flow equal to $\frac{2}{3} d_v$, the wetted perimeter $P_{\frac{2}{3}}$ will be given by the formula

$$P_{\frac{2}{3}} = P - \pi r \qquad (1)$$

For the same depth of flow, the area of the cross-section of flow may be found by subtracting the semicircle $ABDCA$ from the total area A. Hence, for this condition, the area of the cross-section of flow $A_{\frac{2}{3}}$ will be given by the formula

$$A_{\frac{2}{3}} = A - \frac{\pi r^2}{2} \qquad (2)$$

The point M, Fig. 7, is at one-third the vertical diameter BF, and LK is a horizontal line through that point. With the sewer running one-third full, therefore, $KEFGL$ will be the wetted perimeter, and that part of the internal cross-section below the line LK will be the cross-section of the flow. These values, however, cannot be expressed by any convenient general formula; but, when applied to certain cases, simple expressions for them will be given below.

22. Old and New Forms of Cross-Section.—Formerly, it was the general practice to use a value of $\frac{r}{2}$ for the value of r_1. The form of cross-section obtained by using this value of r_1 will here be called the **old form**.

In later years, however, it has become a not uncommon practice to use $\frac{r}{4}$ as the value of r_1. When the flow is light, the latter practice gives a slightly deeper current than the former practice for the same volume of sewage; it is, therefore, a somewhat better practice, especially for large sewers. The form of cross-section in which $r_1 = \frac{r}{4}$ will here be called the **new form**.

23. Elements of the Cross-Section.—The old form for the interior cross-section of an egg-shaped sewer is shown in Fig. 8, and the new form in Fig. 9. The values of all the elements essential to the design of either cross-section, in terms of the upper radius r, are given in Table IV, at the end of this Section. From that table it will be noticed that

the value of the hydraulic mean radius, for either form of an egg-shaped sewer, is greater when the depth of flow is two-thirds the vertical diameter than when the sewer is flowing full. In the old form, the greatest velocity will occur when the depth of flow is approximately .85 of the vertical diameter; and the greatest discharge will occur when the depth of flow is about .93 of the vertical diameter.

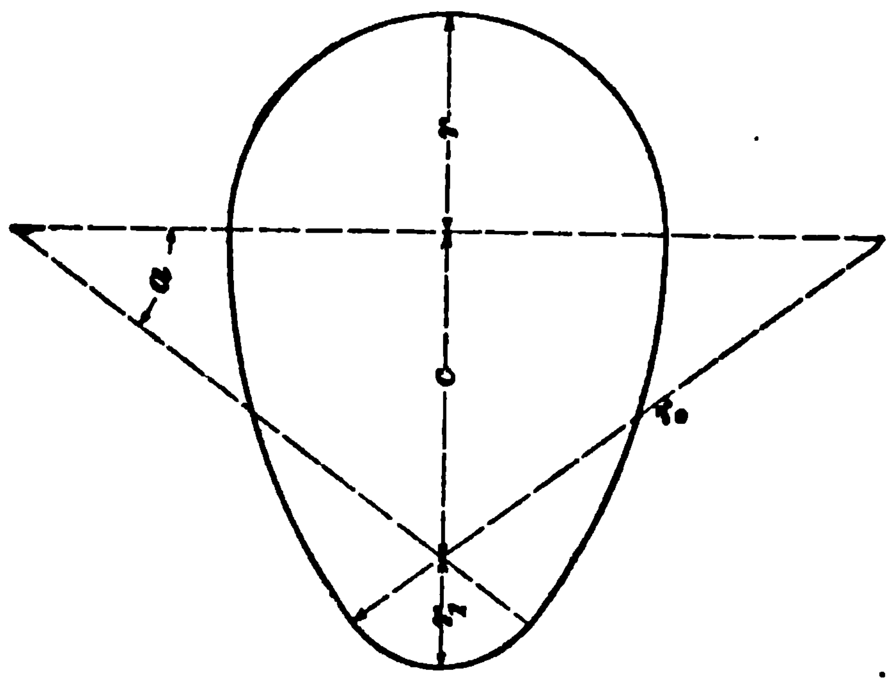


FIG. 8

With the sewer running full, the following approximate formulas will apply to both forms closely enough for most practical purposes:

$$P = \tfrac{21}{8} d_v \qquad (1)$$

$$A = \tfrac{1}{2} d_v^2 \qquad (2)$$

$$R = \tfrac{4}{21} d_v = \tfrac{4}{7} r \qquad (3)$$

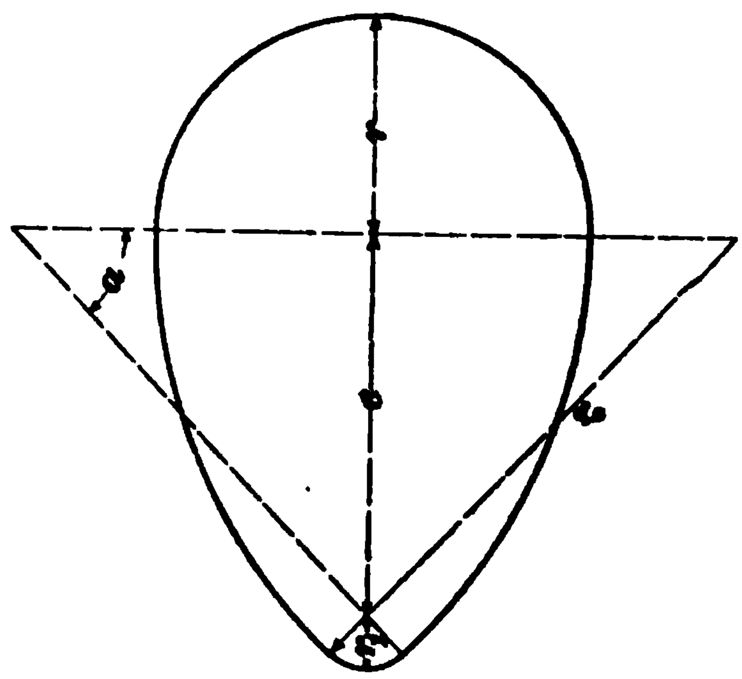


FIG. 9

24. Relative Capacities of Egg-Shaped and Circular Sewers.—If A_c is the internal area of a circular sewer having a diameter equal to the horizontal diameter of an egg-shaped sewer that has an area A, then:

For the old form,

$$A_c = .6838\,A, \text{ and } A = 1.4624\,A_c \qquad (1)$$

For the new form,

$$A_c = .7044\,A, \text{ and } A = 1.4197\,A_c \qquad (2)$$

In computing the dimensions of an egg-shaped sewer, it is generally most expeditious to obtain first the required dimensions of a circular sewer, and then find the dimensions of an egg-shaped sewer having an equivalent discharge.

If r_c is the internal radius of a circular sewer whose area is A_c, then, in order that the areas A and A_c shall be equal, the relative values of r and r_c must be as given by the following formulas:

For the old form,

$$r = .8269\,r_c \qquad (3)$$

For the new form,

$$r = .8393\,r_c \qquad (4)$$

Or, approximately, and near enough for most practical purposes:

For either form,

$$r = \tfrac{5}{6}\,r_c \qquad (5)$$

The same relations will, of course, exist between the horizontal diameters as between the horizontal radii, and, therefore, the three preceding fórmulas will apply to the horizontal diameters by substituting d_h for r and by substituting for r_c the diameter d of the circular sewer.

25. Comparative Values of Hydraulic Radius.— The hydraulic radius for a circular sewer running full is equal to one-fourth the diameter, or $.50r_c$. For the egg-shaped sewer *of equal cross-section*, the hydraulic radius, when the sewer is running full, will be as given by the following formulas:

For the old form,

$$R = .5793 \times .8269\,r_c = .4790\,r_c \qquad (1)$$

For the new form,

$$R = .5688 \times .8393\,r_c = .4774\,r_c \qquad (2)$$

These values are so nearly the same as the value of the hydraulic radius, $.50\,r_c$, of the circular sewer flowing full, that for many practical purposes they may be considered to be the same. Hence, having obtained the diameter of a circular sewer necessary to give a required discharge when running full, the horizontal diameter of an egg-shaped sewer that will give an approximately equivalent discharge may be obtained by applying formula **3** or **4**, Art. **24**, or, near enough for most practical purposes, by applying formula **5**, Art. **24**. If, however, it is desired that the egg-shaped

sewer, when running full, shall have the same ***hydraulic radius*** as the circular sewer when running full, the horizontal diameter d_h of the former must have the value given by the following formulas, in which d is the diameter of the circular sewer:

For the old form,

$$d_h = .863\,d \qquad (3)$$

For the new form,

$$d_h = .879\,d \qquad (4)$$

26. Sewers of Equal Discharge.—In order that the egg-shaped sewer, when flowing full, shall have the same *theoretical discharge* as the circular sewer, its horizontal diameter d_h must have the value given by the following formulas:

For the old form,

$$d_h = .834\,d \qquad (1)$$

For the new form,

$$d_h = .847\,d \qquad (2)$$

The value of the horizontal diameter given by formula **1** is almost identical with that given by formula **5**, Art. **24**, while the value given by formula **2** exceeds that given by formula **5**, Art. **24**, by about 1.6 per cent. Hence, as the practical diameter will be in even inches, and will generally somewhat exceed the theoretical diameter, formula **5**, Art. **24**, will be satisfactory for most practical purposes. When great accuracy is required, however, formulas **1** and **2** may be applied. In applying these formulas to obtain the dimensions of an egg-shaped sewer that will have a capacity equal to that required for a circular sewer, the theoretical or exact diameter, or radius, of the circular sewer should be used.

EXAMPLE.—What must be the upper radius r of an egg-shaped sewer that it may have a cross-section equal to that of a circular sewer 3.64 feet in diameter: (*a*) if the sewer is of the old form? (*b*) if the sewer is of the new form? (*c*) if the sewer is computed by formula **5**, Art. **24**?

SOLUTION.—(*a*) The diameter of the circular sewer is 3.64 ft.; consequently, $r_c = 3.64 \div 2 = 1.82$ ft. Applying formula **3**, Art. **24**,

the upper radius required for an equivalent egg-shaped sewer of the old form will be

$$.8269 \times 1.82 = 1.505 \text{ ft. Ans.}$$

(*b*) By applying formula **4**, Art. **24**, the upper radius required for an equivalent egg-shaped sewer of the new form will be

$$.8393 \times 1.82 = 1.528 \text{ ft. Ans.}$$

(*c*) By applying formula **5**, Art. **24**, the upper radius required for an equivalent egg-shaped sewer is found to be

$$\tfrac{5}{6} \times 1.82 = 1.517 \text{ ft. Ans.}$$

EXAMPLES FOR PRACTICE

NOTE.—In the following examples, three values will be found for the upper radius; namely, (*a*) for the old form; (*b*) for the new form; and (*c*) as given by formula **5**, Art. **24**.

1. What will be the upper radius r of an egg-shaped sewer having the same cross-section as a circular sewer 3.72 feet in diameter?

$$\text{Ans.}\begin{cases}(a)\ 1.538 \text{ ft.}\\(b)\ 1.561 \text{ ft.}\\(c)\ 1.550 \text{ ft.}\end{cases}$$

2. What will be the upper radius r of an egg-shaped sewer having the same cross-section as a circular sewer 2.92 feet in diameter?

$$\text{Ans.}\begin{cases}(a)\ 1.207 \text{ ft.}\\(b)\ 1.225 \text{ ft.}\\(c)\ 1.217 \text{ ft.}\end{cases}$$

3. What will be the upper radius r of an egg-shaped sewer having the same cross-section as a circular sewer 3.20 feet in diameter?

$$\text{Ans.}\begin{cases}(a)\ 1.323 \text{ ft.}\\(b)\ 1.343 \text{ ft.}\\(c)\ 1.333 \text{ ft.}\end{cases}$$

NOTE.—From the foregoing examples, it will be noticed that, for either form of egg-shaped sewer, the value of the upper radius given by formula **5**, Art. **24**, is sufficiently accurate for most practical purposes, being nearly a mean between the values for the old and new forms.

---

27. Practical Dimensions of Egg-Shaped Sewers. In practice, the horizontal diameters of egg-shaped sewers are usually multiples of 2 inches. The practical value of the upper radius r should, therefore, be taken as the nearest full inch above its theoretical value. In case the theoretical value of r obtained for the new form should be expressed by exact inches, it will be well to add 1 inch to its value for its practical value. Having determined the practical value of r, the values of all other dimensions, for either the old or the new form, may be obtained from Table IV at the end of this Section.

Example.—For the example (c), explained in Art. **26**, what will be the value of the following practical dimensions for an egg-shaped sewer of the old form: horizontal and vertical diameters, radii of side and bottom arcs, and vertical distance between centers?

Solution.—In the explanation referred to, the upper radius r was found to be 1.517 ft., or somewhat more than 18 in. Hence, the practical value of r will be 19 in. By applying the coefficients or multipliers given in items 1 to 5, inclusive, of Table IV, the following values are obtained:

$$d_h = 2 \times 19 = 38 \text{ in.} = 3 \text{ ft. } 2 \text{ in.}$$
$$d_v = 3 \times 19 = 57 \text{ in.} = 4 \text{ ft. } 9 \text{ in.}$$
$$r_1 = \tfrac{1}{2} \times 19 = 0 \text{ ft. } 9\tfrac{1}{2} \text{ in.}$$
$$r_0 = 3 \times 19 = 57 \text{ in.} = 4 \text{ ft. } 9 \text{ in.}$$
$$c = 1\tfrac{1}{2} \times 19 = 28\tfrac{1}{2} \text{ in.} = 2 \text{ ft. } 4\tfrac{1}{2} \text{ in.}$$

MATERIALS AND GENERAL CONSTRUCTIVE FEATURES

VITRIFIED TERRA-COTTA PIPE

28. General Use of Terra-Cotta Pipe.—Although sewers have, in times past, been built of stone, brick, concrete, iron, plank, and terra-cotta pipe, this last material is now the only one commonly used for small-sized sewers. It has many important advantages. It is reasonably cheap, and can be made in many parts of the country, so that there are no excessive charges for transportation. When vitrified, it is coated with an impervious vitreous lining on which acids and alkalies, steam and hot water make no impression. The interior surface of a vitrified terra-cotta pipe is so smooth that greater velocities are obtained, for the same grades, than with pipes of other materials. This kind of pipe is so made that it is strong enough to resist the pressure of the surrounding earth and even of a concentrated load, such as a steam roller, on the street surface above it. Sewers do not usually carry sewage under pressure, though the average terra-cotta pipe is able to resist an internal pressure of about 100 pounds per square inch. The weak part of a terra-cotta sewer is the joint, which it is difficult to make water-tight. This feature is treated more fully further on.

29. Making of Terra-Cotta Sewer Pipe.—Sewer pipe as made today contains four ingredients, in the following proportions:

Ingredient	Per Cent.
Fireclay	40
Ordinary yellow clay	15
Sandy loam	40
Old ground-up pipe	5
	100

It is not difficult to grind up and mix together these ingredients. This part of the process is not unlike that used in making brick, the object being to have the ingredients thoroughly pulverized and mixed. The product of the mixture, a stiff clayey mud, is shoveled, or dropped from a conveyer, into a **pipe press.** The principal parts and the operation of a press of this kind are illustrated in Fig. 10: a piston p, driven by a steam piston p' directly above, acts downwards on the clay when steam is admitted above the latter piston in the steam cylinder c. The lower part of the clay cylinder consists of a pipe form f, the size of which can be changed below the line AB. To form the pipe, the clay charge is introduced, and then, by turning on the steam, the clay is forced down into the space s'

Fig. 10

to form the bell of the pipe form f. The diameter of the bell

is dependent on the diameter of the casting *d*, which has to be changed for different sizes of pipe. When the space *s'* is well filled and compacted, as indicated by the clay spurting up through a small orifice *o*, a platform *e* is slowly and automatically lowered, while the clay, continuously under pressure, follows through the annular opening, forming the body of the pipe. The length of the pipe is determined by the distance the platform travels; a pipe of any length could be made, provided the clay charge was great enough, were it not that the damp clay will only hold itself up for a height of a little over 3 feet. On each platform is a piece of plank on which the damp pipe can be set away, after being cut off to the required length. The pipes, each on its board, are then set away to dry for 2 weeks or so. They are then carefully moved and placed in brick kilns shaped like a beehive, about 25 feet in diameter at the base. A coal fire at the bottom of the kiln raises the temperature to about 2,500° F. in about 7 days. The melting point of the aluminum silicate that makes up a large part of the clay is also the point at which sodium chloride, or common salt, breaks up. At this temperature, if a few shovelfuls of salt are thrown into the kiln, the sodium in the salt is set free and combines with the melted silicate on the surface of the pipe, making there a glassy fusion with the pipe itself. The kiln is then shut up, and allowed to cool slowly for about 1 week. The pipes are then removed and are ready for use.

30. Length and Diameter of Sewer Pipe.—Sewer pipes are made of regular and standard dimensions, common to all the pipe factories through the United States. In length, they are made 2, 2½, and 3 feet, the latter being the most desirable, since it reduces the number of joints in the pipe line. In diameter, they are made 4, 5, 6, 8, 9, 10, 12, 15, 18, 21, and 24 inches. All factories carry these sizes in stock. Some factories make other and special sizes, such as 20-inch, and sizes larger than 24, as 27, 30, and 36 inches, the last-mentioned size being the largest found in terra-cotta pipe. Other sizes may be made by special order, but 2 or 3

months are probably required to fill the order, and an additional cost is incurred that will nearly pay for the next larger pipe.

31. Thickness and Strength of Pipe.—Sewer pipes are not made, like water pipe, with a thickness dependent on internal pressure. Their thickness is the result of experience in actual construction and of a number of tests made on pipe of different sizes and makes. The practice of factories is to make pipe of two thicknesses, one known as **standard,** and the other known as **double-strength,** pipe. Table V, at the end of this Section, shows the thickness that well-made pipe should have by the custom of the best factories.

The strength of pipe is a function of the thickness, the thicker pipe being able to carry a greater load without breaking. Tests indicate that standard pipe as made can carry a uniform load of about 2,000 pounds per linear foot of pipe, and double-strength pipe, about 4,000 pounds. The load that sewer pipes must carry is the weight of the earth in the trench above them, with the additional weight of a wagon wheel or a steam-roller wheel, either of which may add 1 ton loading to the pipe. A 12-inch pipe in an 8-foot trench will have a mass of earth $1 \times 8 \times 1 = 8$ cubic feet of earth, or about 1,000 pounds, with 2,000 pounds pressure on the top resting on it. Only a fraction of this loading, however, is transmitted to the pipe, the rest being supported by the sides of the trench. A factor of safety of 3 should be employed. It is safe practice to use double-strength pipe when the pipe is in a trench less than 6 feet deep, and heavy surface loads may be expected. Under other conditions, standard pipe may be used, though double-strength pipe is always safer.

32. Depth of Socket.—The joints in sewer pipe are made by the bell-and-spigot method, as for cast-iron water pipe, the space being filled with cement or composition. There are two types of socket, the **standard** and the **deep-and-wide,** or **deep,** socket. The depths of socket, in inches, are shown in Table VI, at the end of this Section, the depth

referred to being the distance *d* in Fig. 11. The deep-and-wide socket pipe, as supplied by some factories, costs a cent or two more than the standard socket, while other factories furnish either socket at the same price. The advantage of the deep-and-wide socket lies in the fact that the jointing material can be rammed into the sockets to a greater depth, and there is therefore less leakage through the joints. At the same price, deep sockets should always be used in preference to standard sockets, and in wet ground in any case.

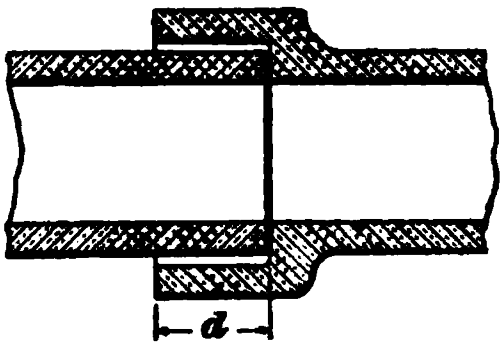


Fig. 11

The width of the socket is not entirely standardized, and although the deep-socket pipe has also greater width than the standard-socket pipe, there is little uniformity. It is usually required by engineers that there shall be a width of socket of at least ½ inch on all sides around the spigot or entering end of the pipe. The factories give this and other amounts.

33. Quality of Sewer Pipe.—In the manufacture of sewer pipe, two extremes should be avoided in burning, the one where the heat is too low, and the other where the heat is too high. In the former case, the vitrifying of the surfaces is not thorough; a broken fragment shows an earthy and yellow section soft enough to be scratched by a knife, and, when a whole pipe is struck with a light hammer, a dull earthy sound is heard instead of the clear ringing report given by a good pipe. If the pipe is overburned, the clay contracts, especially on the ends of the pipe, and cracks appear. If the clay contains lime, and overburning occurs, the lime is calcined, and the gas generated under the glassy coating forms blisters on the surface of the pipe.

Factories are supposed to sell and ship only first-class pipe, unless an inferior grade is bought for the sake of a reduced cost. All pipe should be carefully inspected before acceptance. The following points are to be investigated by an inspector:

1. Whether the thickness of the pipe is that called for by the specifications.

2. Whether the depth of socket and the clear space between the spigot and the inside of the socket, when two pipes are fitted together, are those called for by the specifications.

3. Whether any pipe, intended to be straight, is so warped that the distance on the outside from the middle of the pipe to a taut string from end to end of the pipe is more than $\frac{1}{8}$ inch per foot of length. If so, the pipe should be rejected.

4. Whether any two inside diameters differ by more than one twenty-fourth of the nominal diameter. If so, the pipe should be rejected.

5. Whether there are pieces broken out of the pipe at either end. If so, and at the bell end the broken piece is longer around the pipe than one-half the diameter of the pipe, or extends at all into the body of the pipe; or at the spigot end the broken piece is deeper than $1\frac{1}{2}$ inches, or is longer around the pipe than one-half the diameter of the pipe, or if there is more than one broken piece, so that the pipe cannot be laid with the break on top of the pipe, the pipe may be rejected.

6. Whether there are any unbroken blisters on the inside of the pipe. If so, unless they are less than $\frac{1}{4}$ inch in height, and the pipe can be laid with the blisters on top, the pipe should be rejected.

7. Whether there are any broken blisters on the inside of the pipe. If so, and if they are greater in thickness than one-sixth the nominal thickness of the pipe, or larger in diameter than one-eighth the inner diameter of the pipe, the pipe should be rejected.

8. Whether there are any fire-cracks at either end that extend clear through the pipe. If so, unless they are less than 2 inches long, the pipe should be rejected.

9. Whether the pipe has a clear ringing sound or a dull earthy one when struck with a hammer. If the former, the pipe is probably good. If the latter, the pipe may be underburned or cracked, and should be rejected.

34. Standard Forms of Sewer Pipe.—The common forms of sewer pipe are shown in Fig. 12: (*a*) is a straight pipe, *b* being the bell end and *s* the spigot end; the three curved forms (*b*), (*c*), (*d*) are for turning a right angle, a 45° angle, and a $22\frac{1}{2}$° angle, respectively; the tee branch (*e*) and the wye branch (*f*) are used to make branch connections at 90° and 45°, respectively. There are many other forms into which terra-cotta pipe is made, but their use is restricted to plumbing work, and they are not found in street sewers.

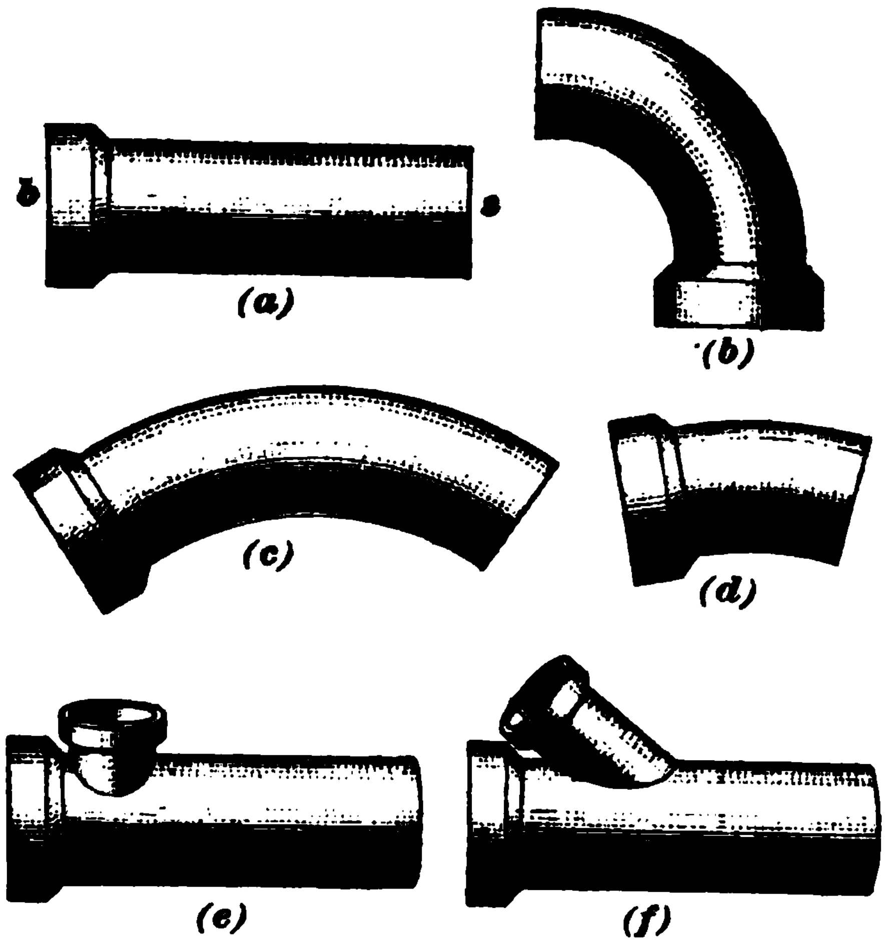


FIG. 12

35. Cement Joints for Sewer Pipes.—The joints in a sewer are the weakest part of the structure, because they allow the line to settle at the joint, and this may separate adjacent lengths of pipe. The joints are not water-tight, both because the jointing material is pervious, and because laborers seldom pack the joint full. For this reason, more or less water comes into the sewer in wet ground; this extra water takes up room, and, if pumping of the sewage is necessary, largely increases the cost. In dry ground, on the other hand, the liquid leaches out and pollutes the soil, leaving the solids behind to choke up the sewer.

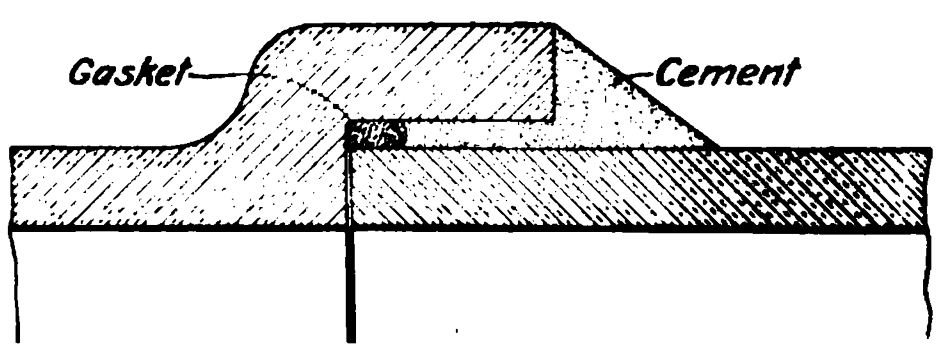


FIG. 13

The usual joint is made with oakum and cement, as shown in Fig. 13. The oakum packing, or **gasket**, is first laid

around the spigot end as the latter enters the bell, and is then tamped back to the bottom of the joint. The rest of the space is then filled with cement mortar, mixed 1 : 1; enough mortar is provided at each joint to have it extend out on the pipe, as shown. This cement should not be soft, but rather moist, so that it can be tamped or rammed into the joint space thoroughly. For the triangular space outside, the cement is best put on directly, with the hands either bare or protected by rubber gloves.

Sometimes, pipes are joined without a gasket, the entire space being filled with cement. In this case, mortar is spread in the socket on the inner surface as it lies in place in the trench. The spigot is then entered as high up as possible, pushed all the way back, and pressed down into the mortar. The remainder of the socket is filled after the pipes are in place. This method is more rapid, but the pipes are less likely to be concentric, and there is more liability that the cement may work up into the inside of the pipe and form miniature dams against the flow.

36. Other Forms of Joints.—Owing to its porosity, cement is not an ideal material for pipe joints. It cannot be run in, but must be placed in by hand, and so may easily be left out of part of the joint. Cement is brittle, and, if there is any settlement, the cement breaks, opening the pipe and causing large leaks or flooding. The substitute that has been most used is a mixture of sand, sulphur, and tar. A joint filled with this mixture is called a **Stanford joint.** The proportions of these ingredients vary, but are generally as follows:

Ingredient	Per Cent.
Sand	50
Sulphur	40
Tar	10

The sulphur is melted in a kettle, the tar is then added, and finally the sand, also heated, is stirred in. The mixture is then ready for use. The joint, in this case, is commonly a bell-and-socket joint having a cross-section like that shown

in Fig. 14. On the end of the spigot end is cast a ring, the outer face of which is a part of a sphere; the casting is made by standing the pipe up vertical inside of a cast-iron mold. The composition is poured into the space between the mold and the pipe to form the ring. Similarly, with an iron mold of another form, the composition is cast into the bell. These preparations are made in the pipe yard or on the side of the trench as the work proceeds. In the trench, the surfaces of the composition are coated with thick oil, the pipes forced together by hand, and the outside of the joint covered with cement. This joint costs more than the cement joint. It is much used in England, and, if properly made, gives much better results than a cement joint.

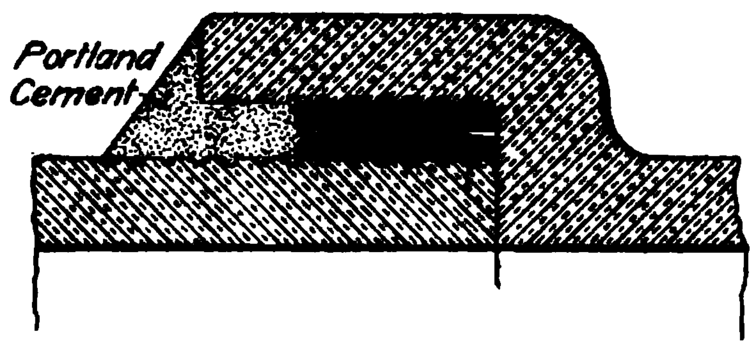


FIG. 14

Probably, some form of asphalt will ultimately be used for joint material. Cloths soaked in asphalt have been calked into pipe joints successfully. The asphalt, however, has to be used hot and in dry pipe—conditions that are difficult to secure in a sewer trench.

SEWERS OF OTHER MATERIALS

BRICK AND CONCRETE SEWERS

37. Cross-Section of Brick and Concrete Sewers. The smaller sizes of sewers, up to 24 inches, are now almost always made circular and of vitrified pipe. Sewers of larger sizes are generally built of brick or concrete, and can be made in any desired form. Where the flow is very variable, there is an advantage in confining the minimum flow to a narrow channel, as this will prevent the accumulation of deposits along the invert. This is accomplished by using the egg-shaped section, which has been fully described in previous articles.

There are other considerations that determine the form of a sewer. For instance, it may be necessary to flatten the section in order to get sufficient curve for the sewer, in which case a section like Fig. 15 may be desirable. Sewers are

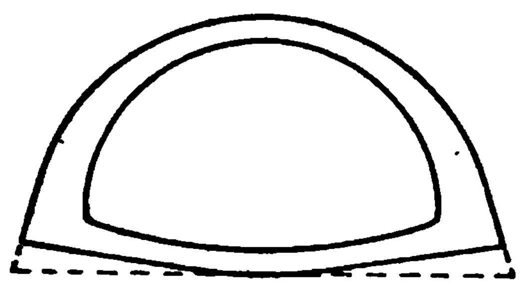


FIG. 15

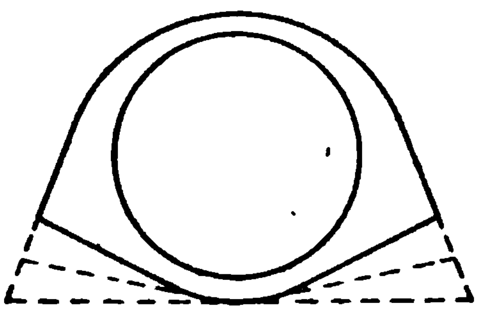


FIG. 16

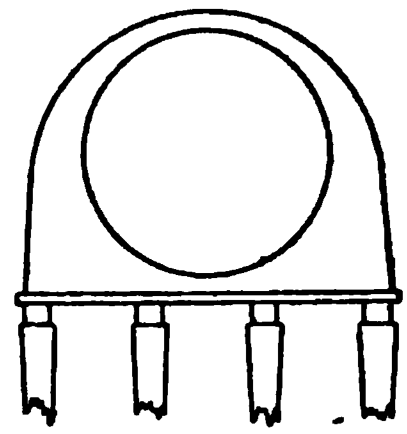


FIG. 17

often built in ground where they must be supported by artificial foundations, and in such cases sections similar to those shown in Figs. 16 and 17 may be advisable, though the relative amounts of masonry should be carefully considered.

38. Quality of Brick for Sewers.—Brick for sewers should be ordinary hard-burned building brick, except for the inner ring of the invert, which should be built of especially hard brick. The hardest bricks from the kiln are selected for this purpose, or paving bricks may be used. Fig. 18 shows the portion of the ring where these special bricks should be employed. All the brick should have a small absorptive power, 2 per cent. being the maximum allowable. The inner ring should be smooth, in order to promote a large flow, and so well-formed bricks, with plain faces and sharp edges, should be insisted on.

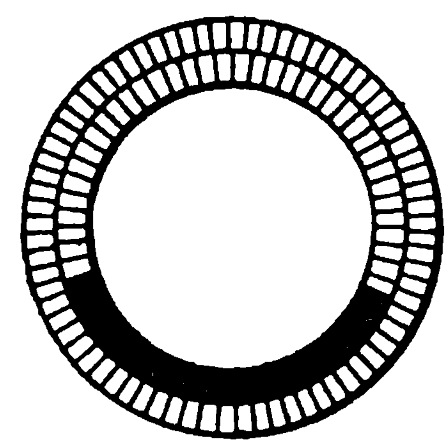


FIG. 18

39. Thickness of Brick Sewers.—The thickness of large brick or concrete sewers is largely a matter of judgment. Theoretically, the number of rings of brickwork, each 4 inches thick, depends on the span and on the loading. The weight of the loading and the proportion of it that reaches the arch of the sewer are so uncertain, and the character of the soil in which the sewer is laid has so great an effect in

holding the sewer together, that no close application of theory is here possible. When the conditions are not unusual, the following empirical formula will generally be found satisfactory for indicating the number of rings required:

$$R = .4 + \frac{D(H-D)}{25},$$

in which R = number of 4-inch rings or courses;
D = internal diameter of a circular sewer, or horizontal diameter of an egg-shaped sewer;
H = total depth of the trench—all in feet.

Any fraction greater than .25 in the value of R should be considered as 1.

The character of the soil, however, is so controlling a factor in determining the size that the judgment of the designer is far more important than the results of the formula. For example, some years ago, in building a 2′ × 3′ egg-shaped sewer in Massachusetts, it was found that in quicksand one course of brick was not enough to hold the ring together, and the sewer had to be built 8 inches thick. On the other hand, in Indiana and Ohio, where the soil is a firm clay, sewers 6 feet in diameter have been built with one ring.

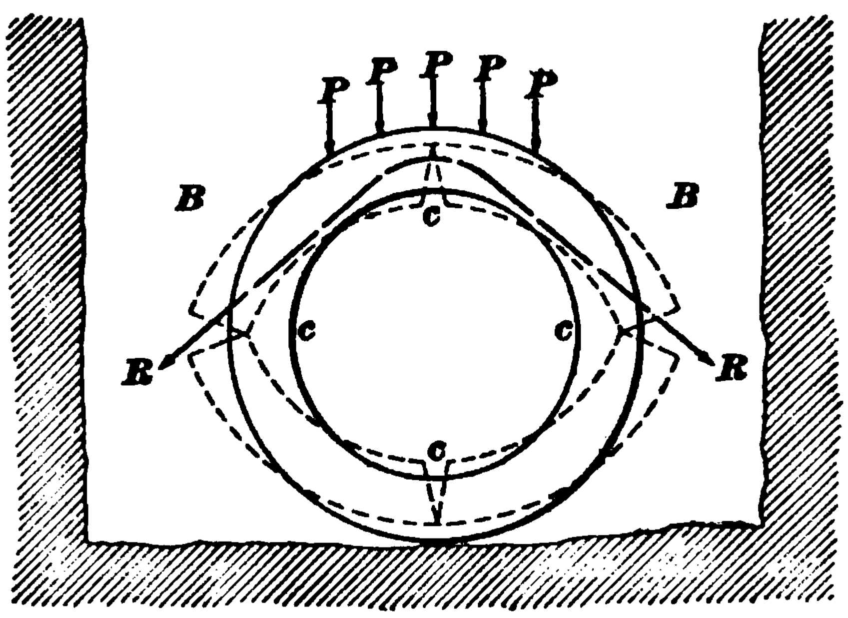


FIG. 19

In ordinary soil, sewers up to 3 feet in diameter may be made one brick thick. From 3 to 6 feet in diameter, they are made two bricks thick. Above 6 feet in diameter, the span and loading become so great that a careful study of the conditions becomes necessary. The weight of the superincumbent earth is transmitted through the brickwork of the arch ring as a pressure between the separate bricks. This may be roughly shown by Fig. 19, in which P, P, etc. represent the pressure of the earth on top of the sewer, and R, R represent the forces acting out from

the sewer into the backing that is shoveled in between the sides of the trench and the sewer. If this backing is not firmly packed in, the sewer has a tendency to break, as shown at *c*, *c*. Owing to the uncertainty as to the action of this backing, no theory can be applied, unless the sewer is assumed to be self-sustaining—that is, so built as not to require any backing. It has been found that four rings are sufficient for the arch.

40. Invert Blocks.—Because in the bottom of an egg-shaped sewer the bricks are required to conform to a great curvature, so that the joints are very thick on the outside, **invert blocks** have been used with success in some cases. These blocks are made of terra cotta, in the form shown in Fig. 20. The surface *b c* forms the invert of the sewer, and the base *a d* rests on a plank set in the trench at the proper grade. These blocks are especially desirable where the new form of the egg-shaped sewer is used.

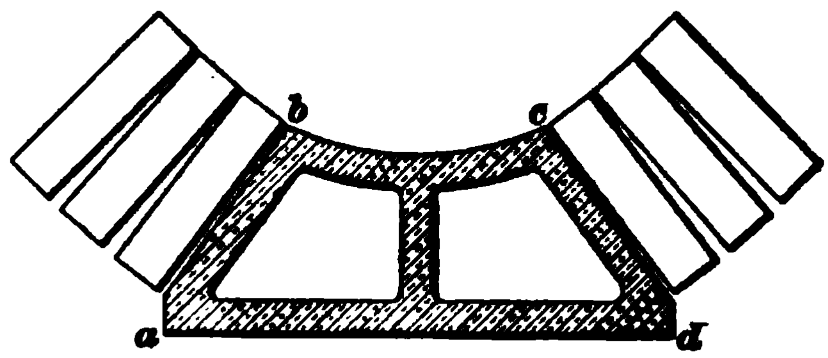


FIG. 20

41. Thickness of Concrete Sewers.—The thickness of the walls of a concrete sewer can be properly determined in only one of two ways, or, better, by a judicious combination of two methods; namely, (1) by an investigation of the thickness of other concrete sewers that have been built and have not fallen down; and (2) by a judgment based on experience in the construction of concrete sewers. Without these two fundamentals, no one is properly qualified to design a concrete sewer. The thickness of circular concrete sewers built in firm and stable ground and at a depth not exceeding 12 feet may be taken to be approximately as follows:

Diameter Feet	Thickness of Sewer Inches
3	4
6	6
9	8
12	10

This thickness must be varied, however, with the character of the soil and the depth of cutting. In wet, running soils, the lower half of the sewer may be from two to four times these thicknesses, with special extra thickness at the sides. In trenches 30 feet deep, the thickness of the arch may be twice the thickness given.

42. Quality of Concrete for Sewers.—The concrete used for sewers should be of first-class quality, carefully proportioned to have as small a percentage of voids as possible. The concrete must be strong, to take up the tensile stresses in the arch; and impervious, to keep ground water out of the sewers. A mixture of 1 : 2 : 4 may be used for the arch, and a mixture of 1 : 2½ : 5 for the bottom. The mixing must be very thorough, and the tamping into place carefully done. For sewer work, the mixture should be so wet that a spade can be readily thrust down into the mass to work the mixture into homogeneity.

CEMENT, WOOD, AND IRON SEWERS

43. Cement Pipe.—Cement sewers have been used in Brooklyn, New York, and are entirely satisfactory when well made. They cost more than terra-cotta pipe, except where freight largely increases the cost of the latter, and consequently are not used today. There is, however, no objection to their use in a town remote from a pipe factory. Since cement pipe is more porous that terra cotta, it is especially adapted for use in dry soil. It can be readily made in molds, which can be bought or made in any size desired.

44. Use of Wood for Sewers.—Wooden sewers are used only under peculiar conditions, since wood is a perishable material and should not be buried under ground. Under water, wood does not decay, and wooden sewers built up like a barrel, with staves hooped together by steel bands, have been used. The design of such pipes has been explained in *Water Supply*, Part 2. The internal pressures in wooden sewers, however, are negligible, and only enough bands are

required to hold the staves in place; they may be spaced at intervals of 3 or 4 feet. Fig. 21 shows the cross-section of a wooden outfall sewer constructed in New London, Connecticut.

45. Use of Iron in Sewer Construction. Cast iron is often used for special purposes, such as for valves and in the form of pipes. Cast-iron pipes are required by railroads, wherever the sewer passes under their tracks, to avoid the possibility of settlement from breakage. They are also used under or over water-supply pipes where there is danger of contamination. They have been used in places where the ground was saturated with water and water-tight sewers were required. In one city, where a brick sewer was found to leak excessively, a wrought-iron pipe was forced through inside from one end, decreasing the diameter slightly, but making a tight sewer. Wrought-iron pipes are not generally used, however, as their life is short under the effects of sewage flow.

Fig. 21

TABLE I

VELOCITY AND DISCHARGE FOR CIRCULAR PIPE SEWERS OF DIFFERENT SIZES ON VARIOUS GRADES FLOWING FULL

(n = .013; Q *in cubic feet per second;* v *in feet per second*)

Diameter Inches	Grade 1 in 10 $s = \frac{1}{10}$		Grade 1 in 20 $s = \frac{1}{20}$		Grade 1 in 30 $s = \frac{1}{30}$		Grade 1 in 40 $s = \frac{1}{40}$		Grade 1 in 50 $s = \frac{1}{50}$		Grade 1 in 60 $s = \frac{1}{60}$	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
6	7.99	1.57	5.65	1.11	4.61	.905	3.99	.784	3.57	.701	3.26	.639
8			7.10	2.48	5.79	2.02	5.02	1.75	4.48	1.57	4.09	1.43
9			7.78	3.44	6.35	2.81	5.50	2.43	4.92	2.17	4.49	1.98
10			8.44	4.60	6.89	3.76	5.97	3.25	5.39	2.94	4.87	2.66
12					7.93	6.23	6.86	5.39	6.14	4.82	5.60	4.40
15							8.12	9.96	7.26	8.91	6.63	8.13
18									8.31	14.7	7.59	13.4
20									8.98	19.6	8.19	17.9
21											8.49	20.4
24												
30												
36												

TABLE I—(*Continued*)

Diameter Inches	Grade 1 in 70 $s = \frac{1}{70}$		Grade 1 in 80 $s = \frac{1}{80}$		Grade 1 in 100 $s = \frac{1}{100}$		Grade 1 in 150 $s = \frac{1}{150}$		Grade 1 in 200 $s = \frac{1}{200}$		Grade 1 in 300 $s = \frac{1}{300}$	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
6	3.01	.592	2.82	.553	2.52	.495	2.06	.403	1.78	.349	1.45	.284
8	3.79	1.32	3.54	1.24	3.17	1.11	2.58	.902	2.23	.780	1.82	.636
9	4.15	1.84	3.89	1.72	3.47	1.54	2.83	1.25	2.45	1.08	2.00	.883
10	4.51	2.46	4.22	2.30	3.77	2.06	3.07	1.68	2.66	1.45	2.17	1.18
12	5.18	4.07	4.85	3.81	4.34	3.41	3.54	2.78	3.06	2.40	2.49	1.96
15	6.13	7.53	5.74	7.04	5.13	6.30	4.19	5.14	3.62	4.44	2.95	3.62
18	7.02	12.4	6.57	11.6	5.87	10.4	4.79	8.48	4.15	7.33	3.38	5.97
20	7.58	16.5	7.09	15.5	6.34	13.8	5.18	11.3	4.48	9.77	3.65	7.97
21	7.86	18.9	7.35	17.7	6.57	15.8	5.36	12.9	4.64	11.2	3.78	9.10
24	8.65	27.2	8.09	25.4	7.24	22.7	5.91	18.6	5.11	16.1	4.17	13.1
30					8.48	41.6	6.92	34.0	5.99	29.4	4.89	24.0
36							7.86	55.6	6.80	48.1	5.55	39.2

TABLE I—(*Continued*)

Diameter Inches	Grade 1 in 400 $s = \frac{1}{400}$		Grade 1 in 600 $s = \frac{1}{600}$		Grade 1 in 1000 $s = \frac{1}{1000}$		Grade 1 in 1500 $s = \frac{1}{1500}$		Grade 1 in 2000 $s = \frac{1}{2000}$		Grade 1 in 3000 $s = \frac{1}{3000}$	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
6	1.25	.246	1.02	.199								
8	1.57	.549	1.28	.446	.982	3.43						
9	1.73	.762	1.40	.620	1.08	.476	.871	.385				
10	1.87	1.02	1.52	.831	1.17	.638	.946	.516				
12	2.16	1.69	1.75	1.38	1.35	1.06	1.09	.856	.936	.735		
15	2.55	3.13	2.08	2.55	1.60	1.96	1.29	1.59	1.11	1.36	.893	1.10
18	2.92	5.16	2.38	4.20	1.83	3.24	1.48	2.62	1.28	2.25	1.03	1.81
20	3.16	6.89	2.57	5.61	1.98	4.32	1.60	3.50	1.38	3.01	1.11	2.42
21	3.27	7.87	2.66	6.41	2.05	4.94	1.66	4.00	1.43	3.44	1.15	2.77
24	3.60	11.3	2.93	9.22	2.26	7.10	1.83	5.76	1.58	4.96	1.27	4.00
30	4.23	20.7	3.44	16.9	2.65	13.0	2.15	10.6	1.86	9.11	1.50	7.36
36	4.80	33.9	3.91	27.7	3.02	21.3	2.45	17.3	2.11	14.9	1.71	12.1

TABLE II

VELOCITY AND DISCHARGE FOR CIRCULAR BRICK SEWERS OF DIFFERENT SIZES ON VARIOUS GRADES FLOWING FULL

(n = .015; Q *in cubic feet per second;* v *in feet per second*)

Diameter Inches	Grade 1 in 75 $s = \frac{1}{75}$		Grade 1 in 100 $s = \frac{1}{100}$		Grade 1 in 200 $s = \frac{1}{200}$		Grade 1 in 400 $s = \frac{1}{400}$		Grade 1 in 1000 $s = \frac{1}{1000}$		Grade 1 in 1500 $s = \frac{1}{1500}$		Grade 1 in 3000 $s = \frac{1}{3000}$	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
24	7.03	22.1	6.09	19.1	4.30	13.5	3.03	9.52	1.90	5.97	1.54	4.84	1.07	3.36
30			7.16	35.2	5.06	24.8	3.57	17.5	2.24	11.0	1.82	8.92	1.26	6.20
36					5.76	40.7	4.07	28.7	2.56	18.1	2.08	14.7	1.45	10.2
42					6.43	61.8	4.54	43.7	2.85	27.4	2.32	22.3	1.62	15.6
48					7.06	88.7	4.98	62.6	3.13	39.4	2.55	32.0	1.78	22.4
54					7.65	122	5.40	85.9	3.40	54.1	2.77	44.0	1.94	30.8
60							5.81	114	3.66	71.8	2.98	58.5	2.09	41.0
66							6.20	147	3.90	92.8	3.18	75.5	2.23	53.0
72							6.57	186	4.14	117	3.37	95.4	2.37	67.0
78							6.93	230	4.37	145	3.56	118	2.50	83.0
84							7.28	280	4.59	177	3.74	144	2.63	101
90							7.62	337	4.81	212	3.92	173	2.76	122
96							7.95	400	5.02	252	4.09	206	2.88	145
102									5.23	296	4.26	242	3.00	170
108									5.42	345	4.42	281	3.12	198
114									5.62	398	4.59	325	3.23	229
120									5.81	456	4.74	372	3.34	262

TABLE III

VELOCITY AND DISCHARGE FOR EGG-SHAPED SEWERS (NEW FORM) OF DIFFERENT SIZES ON VARIOUS GRADES, FLOWING FULL

(n = .015; Q in cubic feet per second; v in feet per second)

Size Inches	Grade .0100		Grade .0070		Grade .0040		Grade .0020		Grade .0010		Grade .0005		Grade .0003		Grade .0001	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
24×36	6.69	29.8	5.59	25.0	4.22	18.8	2.98	13.3	2.09	9.33	1.46	6.51	1.11	4.97		
30×45	7.86	54.8	6.57	45.8	4.96	34.6	3.50	24.4	2.46	17.1	1.72	12.0	1.32	9.16		
36×54	8.94	89.7	7.48	75.0	5.65	56.7	3.98	40.0	2.80	28.1	1.96	19.7	1.50	15.1		
42×63			8.33	114.0	6.29	85.9	4.44	60.6	3.12	42.7	2.19	29.9	1.68	22.9		
48×72					6.90	123.0	4.87	86.8	3.43	61.2	2.41	42.9	1.85	33.0	1.03	18.3
54×81							5.28	119.0	3.72	84.0	2.61	59.0	2.01	45.3	1.12	25.3
60×90							5.67	158.0	4.00	111.0	2.81	78.3	2.16	60.2	1.21	33.7
66×99							6.04	204.0	4.26	144.0	3.00	101.0	2.31	77.9	1.30	43.8
72×108							6.40	257.0	4.52	181.0	3.18	128.0	2.45	98.4	1.38	55.5
78×117							6.75	318.0	4.77	225.0	3.36	158.0	2.59	122.0	1.46	69.0
84×126									5.01	273.0	3.53	193.0	2.72	149.0	1.54	84.3
90×135									5.24	329.0	3.69	232.0	2.85	179.0	1.62	102.0

TABLE IV

ELEMENTS OF CROSS-SECTION OF EGG-SHAPED SEWERS

Element	Symbol	Value for Old Form	Value for New Form
1. Horizontal diameter	d_h	$2r$	$2r$
2. Vertical diameter	d_v	$3r$	$3r$
3. Radius of bottom arc	r_1	$\frac{1}{2}r$	$\frac{1}{4}r$
4. Radius of side arcs	r_0	$3r$	$2\frac{2}{3}r$
5. Distance between centers . .	c	$1\frac{1}{2}r$	$1\frac{3}{4}r$
6. Distance $OC = O'C$ (Fig. 7)		$2r$	$1\frac{2}{3}r$
7. Wetted perimeter, full . . .	P	$7.9299r$	$7.8409r$
8. Wetted perimeter, $\frac{2}{3}$ full . . .	$P_{\frac{2}{3}}$	$4.7883r$	$4.6994r$
9. Wetted perimeter, $\frac{1}{3}$ full . . .	$P_{\frac{1}{3}}$	$2.7493r$	$2.6651r$
10. Area of flow, full	A	$4.5941r^2$	$4.4602r^2$
11. Area of flow, $\frac{2}{3}$ full	$A_{\frac{2}{3}}$	$3.0233r^2$	$2.8894r^2$
12. Area of flow, $\frac{1}{3}$ full	$A_{\frac{1}{3}}$	$1.1364r^2$	$1.0171r^2$
13. Hydraulic radius, full	R	$.5793r$	$.5688r$
14. Hydraulic radius, $\frac{2}{3}$ full . . .	$R_{\frac{2}{3}}$	$.6314r$	$.6148r$
15. Hydraulic radius, $\frac{1}{3}$ full . . .	$R_{\frac{1}{3}}$	$.4133r$	$.3817r$
16. Angle COC' (Fig. 7)	a	$36° 52' 12''$	$46° 23' 50''$
17. Angle $EC'G$ (Fig. 7)	b	$106° 15' 37''$	$87° 12' 22''$

TABLE V

THICKNESS OF SEWER PIPE

Kind of Pipe	Diameter, in Inches										
	6	8	9	10	12	15	18	21	24	30	36
Standard	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{5}{8}$	2	$2\frac{1}{4}$
Double-strength .					1	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{2}$	3

TABLE VI

DEPTHS OF SOCKETS FOR STANDARD AND FOR DEEP-AND-WIDE SOCKET

Kind of Socket	Diameter, in Inches										
	6	8	9	10	12	15	18	21	24	30	36
Standard	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2	$2\frac{1}{4}$	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{1}{2}$	3	$3\frac{1}{2}$
Deep-and-wide . .	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	3	$3\frac{1}{4}$	$3\frac{1}{2}$	$3\frac{3}{4}$	4	$4\frac{1}{2}$	5

SEWERAGE

(PART 3)

SEWER CONSTRUCTION

MANHOLES AND LAMP POLES

1. Use of Manholes.—A **manhole** is an opening leading from the surface of the ground to a sewer, and used for purposes of inspection and cleaning. An inspector can get down a manhole, and, by looking through the pipe to a light at the next manhole, tell whether the pipe is obstructed or not. If it is, he may have workmen run scrapers and brushes through it until the obstruction is removed.

2. Location of Manholes.—In order that workmen may use manholes for cleaning purposes, the distance between manholes must not exceed certain limits. For small-pipe sewers, the manholes may be placed 300 to 400 feet apart, that being about the distance which a stick can be forced. On larger sizes—24 inches or more—the manholes may be 600 feet apart. Their position is also determined by the requirement that there shall be a manhole at every junction of two or more sewers, at every change of grade of every sewer, and at every change of direction. This makes it necessary that the sewer should run in straight lines from manhole to manhole, and that the changes in line and grade should come at or in the manhole. Since sewer crossings usually occur at street intersections, manholes are generally placed at such intersections, and, in long blocks, midway between.

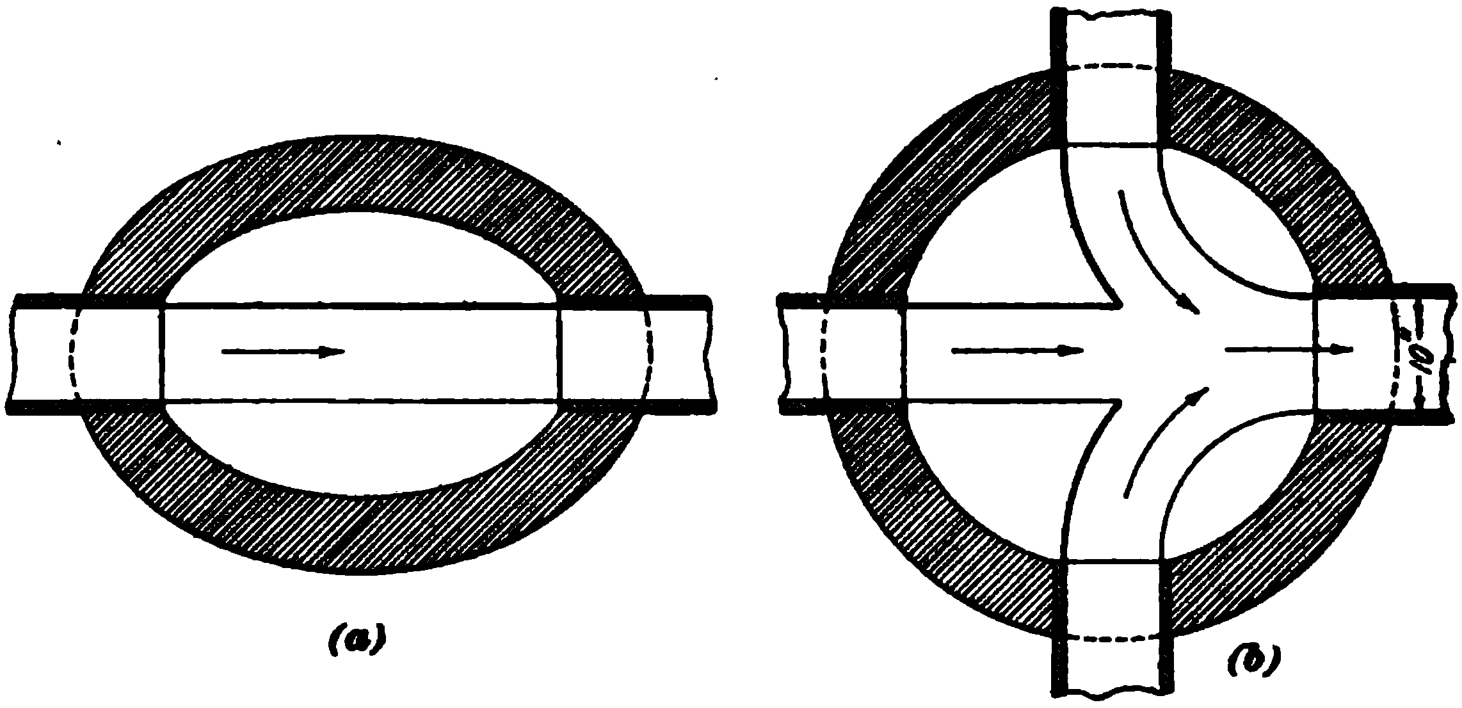


FIG. 1

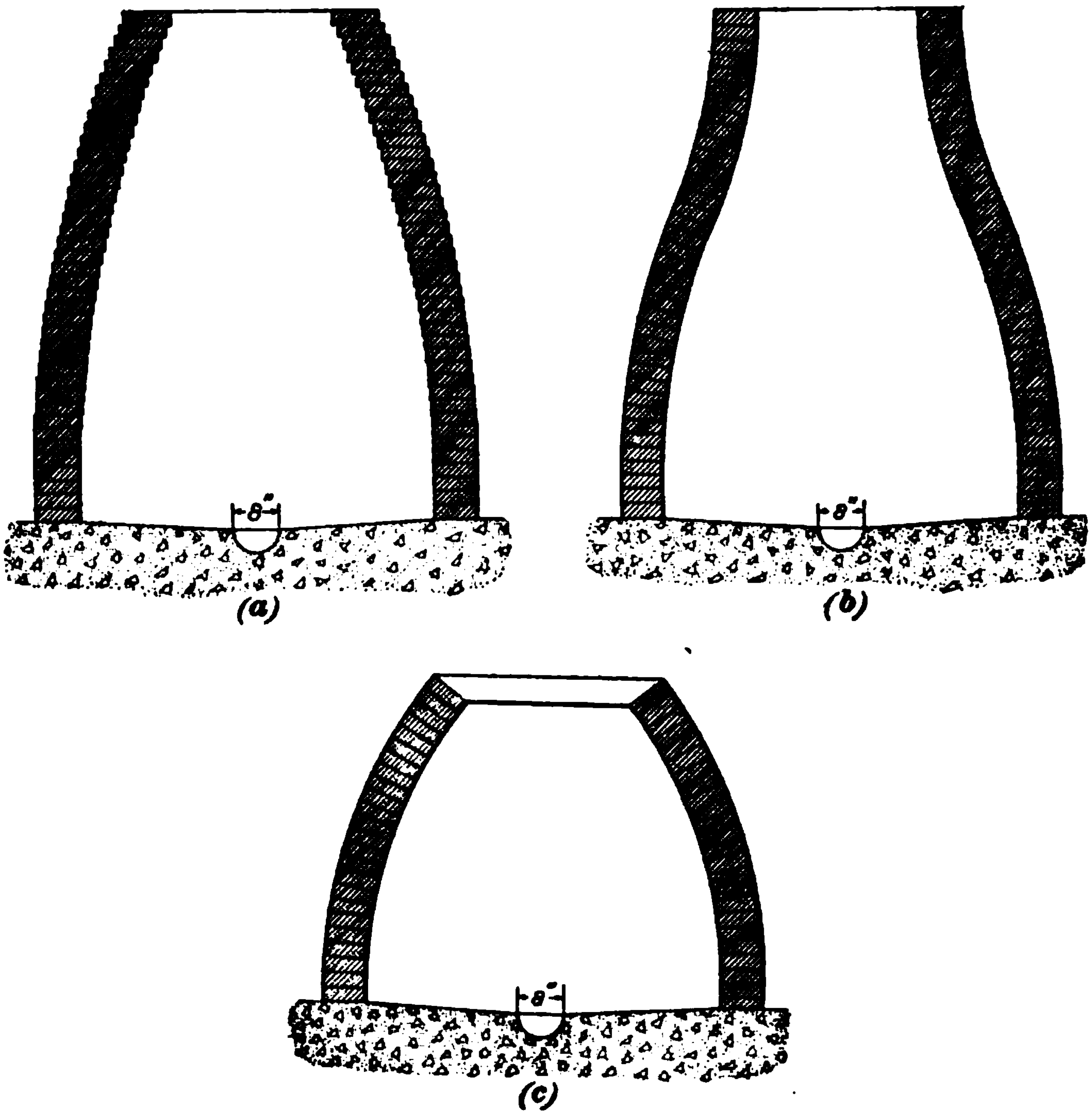


FIG. 2

When the street grade breaks within the block, a manhole is usually added, no matter what the length of the block may be.

3. Shape of Manholes.—Manholes are usually made bottle-shaped, the inside diameter at the top being almost always 24 inches, and the diameter at the bottom depending on the size of the sewer. For 6- and 8-inch sewers on a straight line, the bottom is oval and 4 ft. × 3 ft., as shown in Fig. 1 (*a*). Where an intersection occurs, the bottom is circular and is 4 feet in diameter, as shown in Fig. 1 (*b*). The latter figure shows a 4-foot manhole at a point where two laterals enter the main line, and illustrates how changes of direction are made in the manhole.

The vertical section of the manhole may be any one of the three shapes shown in Fig. 2. In (*a*), the bricks are laid horizontal, the reduction in size at the top being made by offsetting each course of bricks just enough to secure the desired reduction in diameter. In (*b*), the bricks are kept normal to the inside walls. In (*c*), which illustrates a form used only for shallow manholes, 4 feet deep or less, the courses are kept normal, as in (*b*), but are not brought back to horizontal positions. The form (*a*) requires the most bricks, and has more room inside. It should be used for depths between 5 and 10 feet.

4. Bottom and Section of Manhole.—The bottom of a manhole may be made either of brick or of concrete, the

FIG. 3

latter being the simpler construction. Fig. 3 shows how a brick bottom is started. The ends P, P_1 of the pipes are left the proper distance apart, protruding from the bank

of dirt, which is thrown back into the trench. A row of bricks on edge is first laid from A to B, the tops forming a continuous line from the invert of the pipe P to the invert of the pipe P_1. Where a change of grade occurs, this line of bricks becomes a vertical curve. Against this line, another row of bricks is laid, being held up on the back by dirt tamped in and by a layer of mortar. These rows are carried up on each side to the horizontal diameter, conforming to the curvature of the pipe. Beyond that, they rise vertically to the top of the pipe, as shown in section in Fig. 4. The inside wall of the manhole is brought just flush with the

FIG. 4

ends of the pipes, and a brick floor is carried around on the refilled earth.

With concrete, a mass is dumped down in the bottom of the manhole excavation, leveled up for a floor at about the level of the top of the entering pipes, and a channel formed in the concrete by a trowel and by hand. A wooden form is also used against which the concrete is packed; but a good workman using damp concrete can shape it without the form. Sometimes, a split pipe—that is, a pipe split longitudinally (easily done with a cold chisel, when needed)—is put in through the manhole, and the concrete packed in against it. The concrete should extend outwards to support the manhole walls, and project about 6 inches beyond them.

5. Thickness of Walls.—There is no theory available for determining the thickness of manhole walls. Under ordinary conditions, they are two bricks, or 8 inches, thick.

For shallow manholes built in dry clay, a 4-inch wall may be built, but it is not a safe construction. Where a high head of ground water is encountered, where heavy lateral pressure is expected, and where the manhole extends more than 15 feet below the surface, the walls may be made 12 inches thick, but this is rarely necessary.

6. Manhole Frames and Covers.—The walls of a manhole are capped with a cast-iron frame and cover, the latter

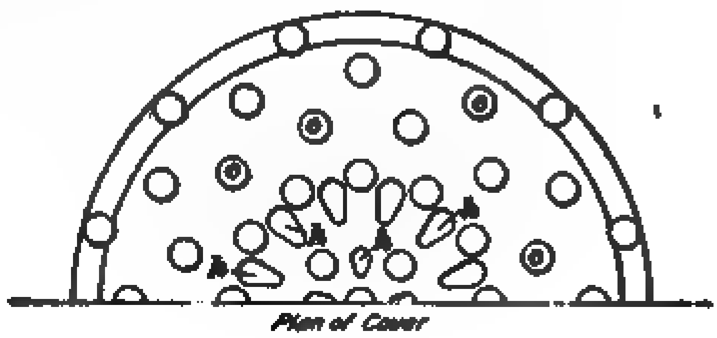


FIG. 5

being flush with and a part of the street pavement. The frames are usually 8 inches high, to allow paving blocks to be laid in on top of the brickwork and flush with the cover. This height may be reduced on brick or asphalt streets. Fig. 5 shows a frame *F F* and a cover *C C*. The base *B B* of the frame is from 4 to 5 inches wide, and rests on a bed of mortar directly on the brickwork. The thickness of the cast iron in the frame is generally 1 inch. The holes *h* in the

cover are used for ventilation, and the projections *c* to prevent the top from wearing smooth and becoming slippery. The frame weighs about 200 pounds, and the cover about 150 pounds. They may be designed by the engineer and cast in a local foundry, or may be bought in standard designs from large foundries.

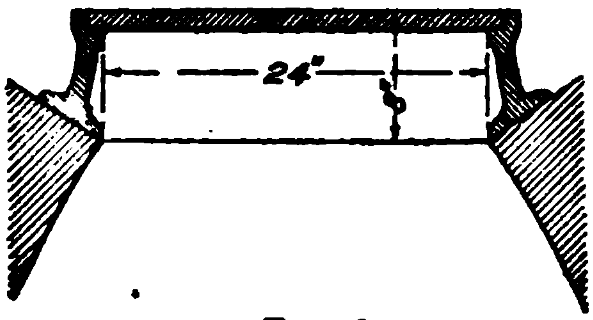


FIG. 6

For the manhole shown in Fig. 2 (*c*), a special frame, such as is shown in Fig. 6, must be designed, to fit the inclined brickwork and have the top horizontal. The patterns for such a casting are expensive, and, if possible, it is better to avoid this construction.

7. Special Size of Manholes.—On 18- to 36-inch sewers, the base of manholes may be 6 or even 8 feet in diameter, in order to make the curves of the sewers inside the manhole. It is best to draw the lines of pipe meeting at the manhole, sketch in the curves necessary to make the flow lines meet smoothly, and make the manhole walls of a sufficient diameter to include all the curves. Fig. 7 shows a junction of two 24-inch and one 18-inch sewer with a 36-inch sewer. The curves must not be too sharp, and the tongues t_1, t_2 must project into the main sewer far enough to guide the flow into the main line without any eddying or other disturbance. The large circle represents the inside walls of the manhole, and is 7½ feet in diameter. As will be explained further on, it is sometimes necessary to have manholes of extra size, which will contain

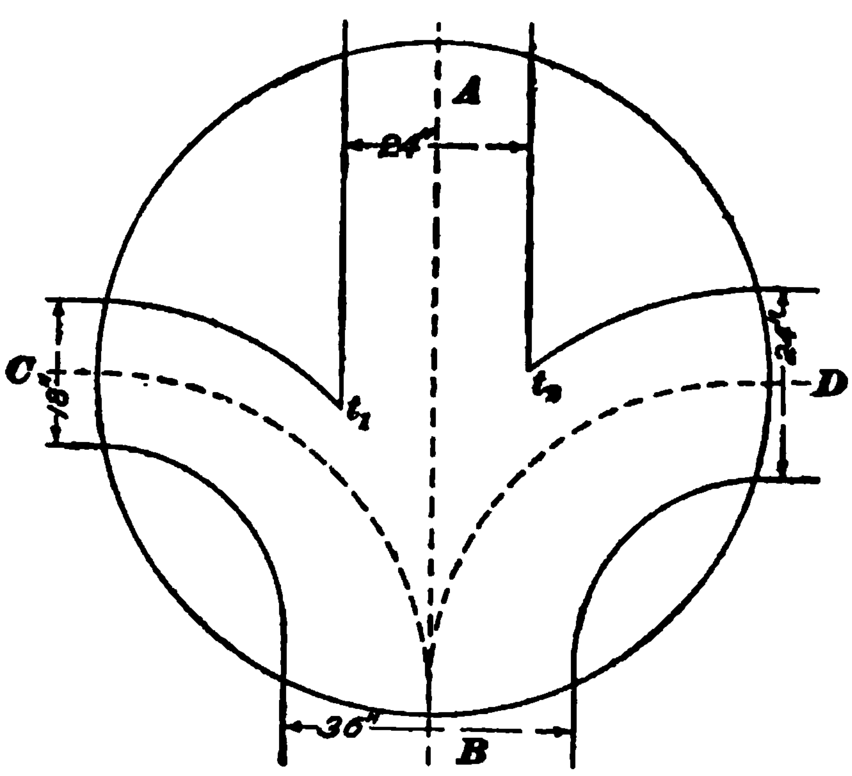


FIG. 7

valves or gates, or allow the making of connections between sewers on different grades.

8. **Connections Between Sewers on Different Grades.**—Where the connecting sewers, as shown in Fig. 7, are at the same level, the connections are easily made; but it frequently happens that, on account of topography, the main sewer *A B* is deep at its upper end while *C* and *D*, perhaps extending across the valley, need to be only a few

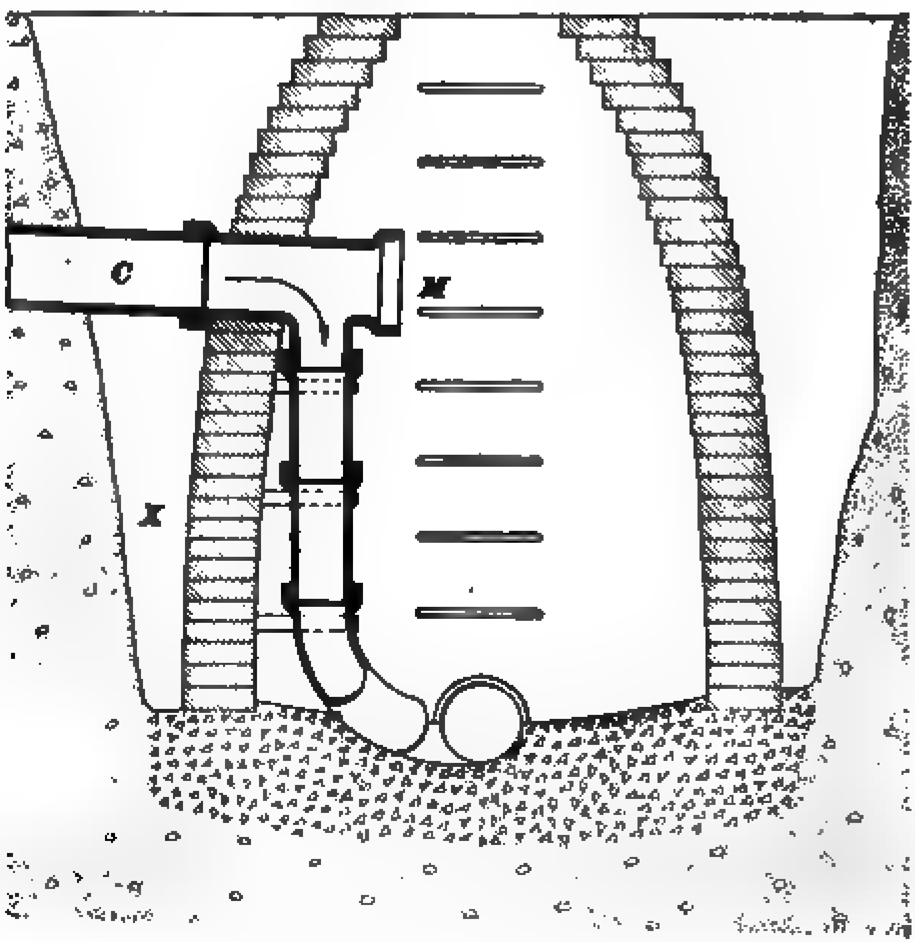


FIG. 8

feet deep. The problem then is to connect the flow of *C* and *D* smoothly with the flow of *A B*.

Fig. 8 shows on a smaller scale than Fig. 7 the usual construction for these conditions. The pipe line *C* ends in a T pipe so placed that the T branch comes just inside the manhole wall. Vertical pipes then carry the flow down and around a bend, which can be about 30°, to the main sewer. The elbow is built into the bottom of the manhole, and the flow line continued in the brick or concrete. This construction

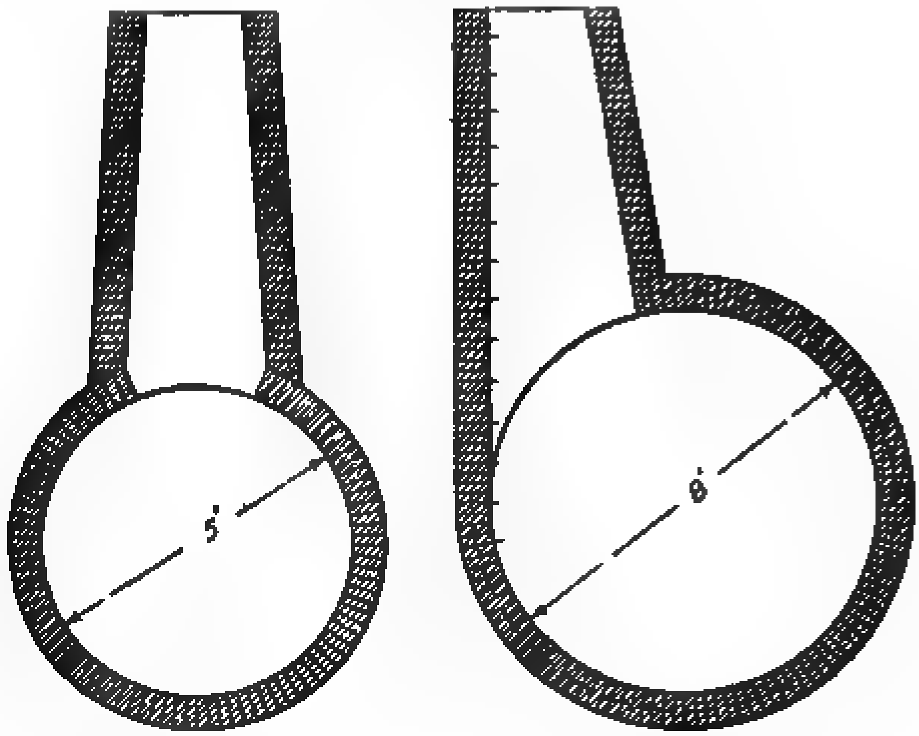
is faulty if the space X is so large as to allow the pipe C to settle and break away from the T. Special care must be used in tamping the earth back in the hole X; if obtainable, sand settled in water is the best material for such purpose. The opening M serves for inspection and cleaning.

9. For sewers larger than 10 inches, especially if two laterals meet in the same manhole, the preceding construction would take up too much room in the manhole, or necessitate a very large manhole. Under such conditions, the vertical pipes are brought down on the outside, as shown in Fig. 9. As before, the T pipe C must be carefully supported and the elbow at the bottom should be buried in concrete to carry the weight of the vertical pipe. From the lower end of the elbow E to the main sewer S, the channel is made in the concrete.

FIG. 9

10. Manholes for Large Sewers. Sewers 3 feet or more in diameter are large enough to be entered, and curves of long radius may be used. Manholes are necessary only to provide access to the interior, and will generally rest with their walls directly on the arch of the sewer. For sewers up to about 6 feet in diameter, the manhole should be symmetrical with respect to the center line of the sewer, as shown in Fig. 10 (a); for larger diameters, the manhole may be built on one side, with one wall of the manhole tangent to the side of the sewer, as shown in Fig. 10 (b).

11. Lamp Holes and Fresh-Air Inlets.—For pipe



(a) Fig. 10 (b)

sewers, where the distances between manholes is considerable, the means of observing the condition of the sewer at intermediate points may be afforded by what are called **lamp holes.** A lamp hole is formed by placing a **T** in the line of the sewer and carrying a vertical pipe to the surface, as shown in Fig. 11. The sewer may be examined by means of a lamp lowered through the vertical pipe.

If the lamp holes are carried to the surface and covered with perforated cast-iron covers, they will assist materially in the ventilation of the sewer. They are then commonly called **fresh-air inlets.** Lamp holes and

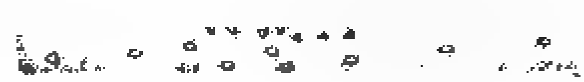


Fig. 11

fresh-air inlets can be cheaply constructed, and afford means for inspecting the condition of pipe sewers, but are not available for cleaning the sewer or removing obstructions. They have the advantage of preserving continuous and uniform the sectional form of the lower part of the sewer, thus avoiding deposits.

12. In paved streets, the top of the vertical pipe should be covered and protected with a heavy cast-iron frame and cover, such as is shown in Fig. 12. In unpaved streets, the

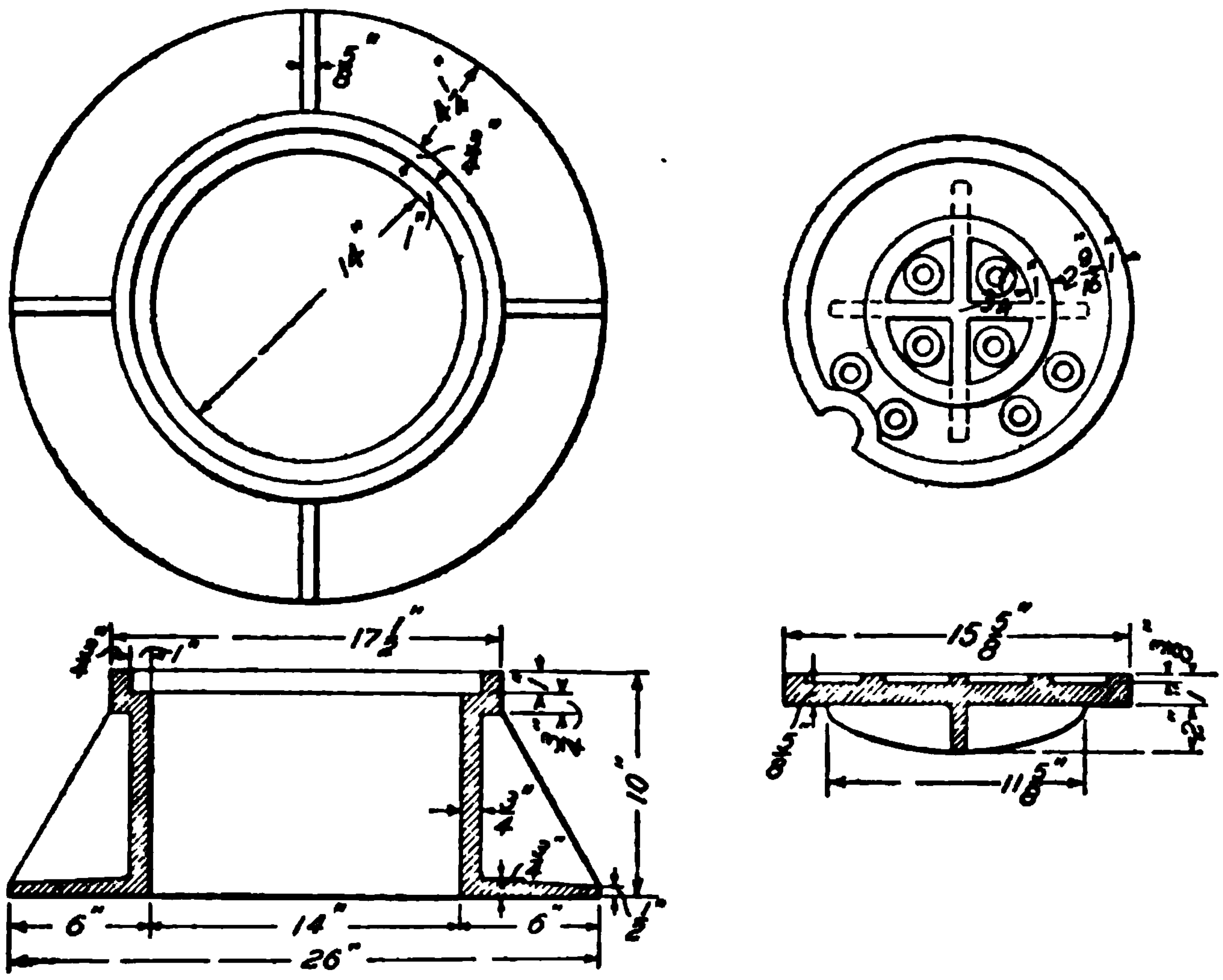


FIG. 12

top should be left about 1 foot below the surface, covered with a light casting or with a terra-cotta cap. In all cases, the **T** should be well supported and buried in a mass of concrete, to prevent the weight of the vertical pipe from breaking it. The concrete therefore should be brought well up on the pipe, as shown in Fig. 11. The iron casting protecting the pipe should be supported separately on a bed of concrete, and the pipe should be free to settle independently; that is, the concrete supporting the casting should not touch the pipe, but should be supported at least ½ inch all around.

CATCH BASINS

13. Catch basins are placed at street corners and, sometimes, if the block is long, at intermediate points also. They are for the purpose of admitting storm water to the sewers, and, therefore, are used in the combined system only. Catch basins are built in various forms, most of which, however, consist of a chamber or basin, into which the storm water flows directly from the street gutters, having an outlet into the sewer from a point at some distance above the bottom. By this arrangement, a large proportion of the coarsest and heaviest of the matter suspended in the storm water will not enter the sewer, but will settle to the bottom of the catch basin and be retained there. In order to prevent sewer gas from entering the catch basin (from which it would escape in the vicinity of the sidewalk and be very objectionable), the outlet to the sewer is given such a shape as to form a trap.

Fig. 13

The form of catch basin most common in the United States is shown in Fig. 13. It is generally built of brick or concrete. When built of brick, it should be lined with cement and plastered both inside and outside with cement mortar, so that it will not leak. In the figure, *o* is the opening for admitting the storm water from the gutter; *t* is the trap to the outlet leading to the sewer *s*; *w* is the surface of the water, and *m* is the deposit of mud and sedimentary refuse; *c* is a cast-iron cover to an opening in the top of the catch basin, through which the deposited mud and refuse may be removed.

It is important that catch basins should be perfectly watertight, so that the water surface may be kept at the proper level. This is necessary, in order both to cover the deposit

of mud and prevent it from giving off disagreeable gases, and to seal the trap against the escape of sewer gas. The water level in street catch basins should be from 2 to 3 feet below the street surface, and the total depth of the basin should generally be from 6 to 8 feet, in order to avoid freezing. The construction may be varied in detail to suit conditions.

FOUNDATIONS

14. Foundations for Pipe Sewers.—Usually, small pipe sewers do not require foundations other than the natural earth at the bottom of the trench. In the case of large sewers in heavy cuttings or in soft and treacherous soil, an artificial foundation may become necessary. Just when such foundations should be used cannot be made a matter of formula or theory. Aside from experience, the only guide is to take a stick of timber a foot square, stand it on end in the trench, and load it with two or three times the load that the pipe may be expected to carry. If the settlement is continuous, some foundation should be provided.

The simplest method of dividing up the load is to lay in the bottom of the trench 1″ × 12″ boards, end to end, and joined by a splice board underneath. This is sufficient, for pipes up to 12 inches in diameter, to carry the weight over soft spots and to prevent uneven settlement at joints.

Fig. 14

If the bottom of the trench becomes soft during excavation, so that the bottom grading and position of the pipe are uncertain, 4 or 5 inches of the mud should be removed, and gravel substituted. If the pipe is larger than 12 inches, and the cutting is deep, concrete is used in which to bed the pipe, in the manner shown in Fig. 14. Care should be taken in the use of concrete to remove all soft mud from the bottom of the trench, or it will be forced up into the concrete. Also,

a fresh layer of concrete should be deposited in the trench before the pipe is put in place.

15. Foundations for Brick Sewers.—Ordinarily, a brick sewer is built directly on the natural soil, tamping in the earth behind in space *C*, Fig. 15, as the courses of brick rise from the invert on each side. This is not, however, a very firm construction, since the dirt replaced cannot be tamped and is very likely to settle and distort the cross-section of the sewer.

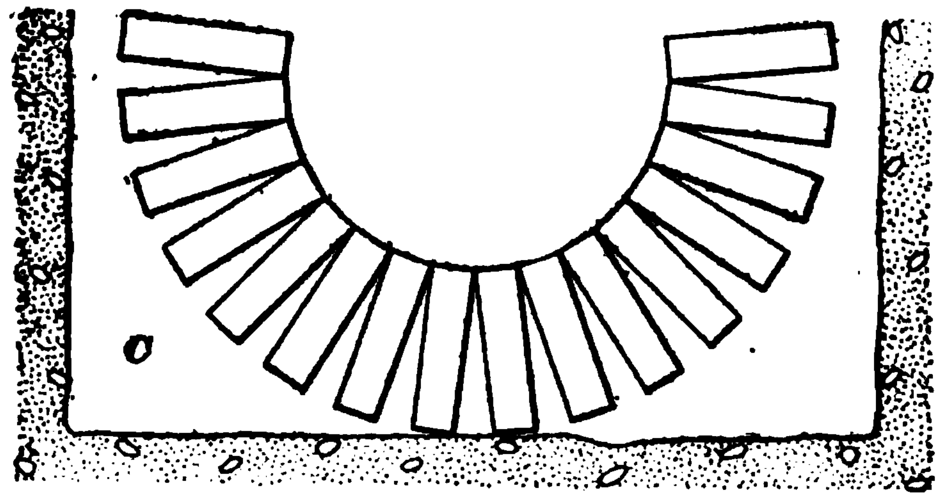


FIG. 15

For this reason, especially if the ground is soft, a **cradle** is often used. This is a wooden structure, so built as to rest securely in the bottom of the trench and carry the brick on its interior surface. It is made up on a series of U frames cut out of 2″ × 8″ or 2″ × 12″ plank shaped and spiked together, as shown in Fig. 16. The diameter of the inside of the cradle should be that of the outside of the sewer, so that the bricks can be laid directly against the wood. These forms are spaced about 24 inches apart, and the cradle sections are usually 8 feet long. They are made up on the bank, lowered to place in the trench, and blocked up to grade. Then, earth is tamped in solidly behind the lagging, and the cradle is ready for the bricks.

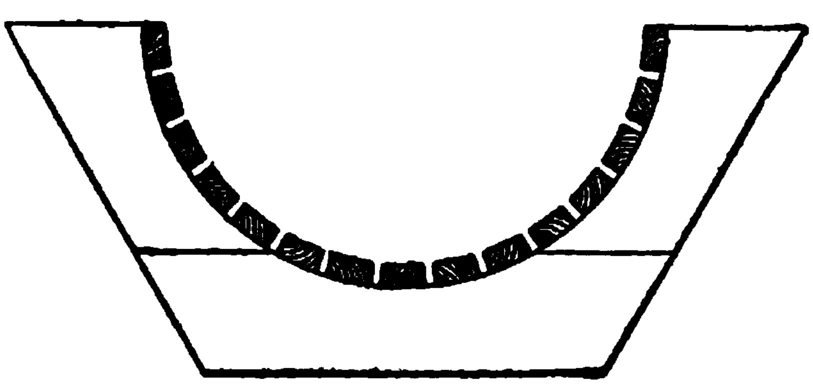


FIG. 16

16. A cradle should not be used in a dry trench nor in a trench that is alternately wet and dry, because the wood will decay and allow the sewer to settle. For these conditions, concrete must be used. Fig. 17 shows a suitable section of a concrete sewer foundation constructed in Brooklyn, New York.

14

FIG. 37

17. Timber Foundations.—If the ground is very wet and unstable, a platform may first have to be built under the

Fig. 18

concrete. Longitudinal timbers about 4 in. × 8 in. are laid down, and a platform is built on them, much like a wooden

Fig. 19

sidewalk. If more than this is needed, the longitudinal timbers are supported on piles, which are also occasionally used

in the case of pipe sewers. Fig. 18 shows the construction for pipe sewers; Fig. 19, the construction for a circular brick

Fig. 20

sewer with concrete invert; and Fig. 20, the construction for an egg-shaped sewer—all being common sizes, and showing standard practice.

CONSTRUCTION OF BRICK AND OF CONCRETE SEWERS

18. Details of Construction of Brick Sewers.—Brick sewers are generally laid with all bricks as stretchers, making rings about 4 inches thick, each ring being carried around separately and keyed at the top of the arch. When it is possible, however, headers should be inserted to tie the two arch rings together, as shown at *B*, Fig. 21. The bricks in the invert are laid to lines stretched on templets made for the number of rings required. The arch is laid over tightly boarded forms, which are withdrawn and advanced as soon

as practicable. The collar joint (the mortar joint between separate rings) and the outside plastering are mainly to be depended on to exclude water, as the bricks are generally porous.

Bricks should never be laid in contact, but should always be bedded in mortar. In sewer work, it is better to lay them with what is called a *push joint* or a *shove joint*, in which mortar is first spread thickly and the brick then pushed into place until the mortar flushes out. Bricks should be wet before they are laid, as otherwise they will absorb moisture from the mortar, and the latter will not set properly.

FIG. 21

Brick sewers are generally joined by long curves, and, as it is somewhat difficult to build them smoothly around curves, they are usually built in very short straight sections, the invert being tested as to its form by sliding a templet along it. It is customary to increase the grade slightly around curves, due allowance being made when fixing grades on the profiles.

19. Details of Plain- and of Reinforced-Concrete Sewers.—The invèrt of concrete sewers is formed by ramming the concrete into place on the bottom of the trench and bringing the interior of the invert in line with templets substantially fastened in the trench. Sometimes, these templets are in the form of the cross-section of the invert, and the invert is formed by moving along them a straightedge, which cuts off any excess of concrete. Another method is to set longitudinal guide pieces about level with the horizontal axis of the sewer, and to slide a templet of the proper form along them.

The invert is usually formed slightly larger than the finished sewer, to allow for a coat of cement mortar, which should be applied to it and smoothly troweled before the

FIG. 22

FIG. 23

construction of the upper arch. The upper arch is tamped in place over smoothly boarded forms. These should be substantially made and rigidly supported, so that when the concrete is tamped over them they will not spring and so prevent a proper bond in the concrete.

20. If steel-wire netting, or metal in other forms, is built into the concrete, the steel will add greatly to the strength and stability of the sewer, particularly in unfavorable soils. When this is done, the quantity of concrete may often be reduced enough to more than pay for the reinforcement. This construction is shown in progress in Fig. 22, which is a photograph from actual work. The netting is seen folded back on each side in the foreground, ready to be brought up on the forms as soon as they are ready. Its position in the concrete is shown by the dotted line.

Fig. 23, taken from "Engineering Record," shows another method of combining steel and concrete, the steel being in the form of rods, which are inserted at intervals of 15 inches around the sewer, and longitudinally as well.

STREAM AND OTHER CROSSINGS

21. Inverted Siphons.—Very often, main sewers are laid along the course of small streams that follow, in an irregular way, depressions toward which all the smaller sewers in the area tributary to the stream must flow. Such streams generally cross and recross the valley, flowing first along the foot of one bluff and then along that of another. It is frequently better, and sometimes necessary, to locate the main sewer along a more direct line than that of the stream, and this requires that the sewer should cross the stream. It is also sometimes necessary to make such crossings on branch sewers, in order to discharge them into a main sewer that lies on the opposite side of a stream. Occasionally, these crossings can be made without depressing the grade of the sewer, but generally the grade must be depressed, forming what is called an **inverted siphon.** In

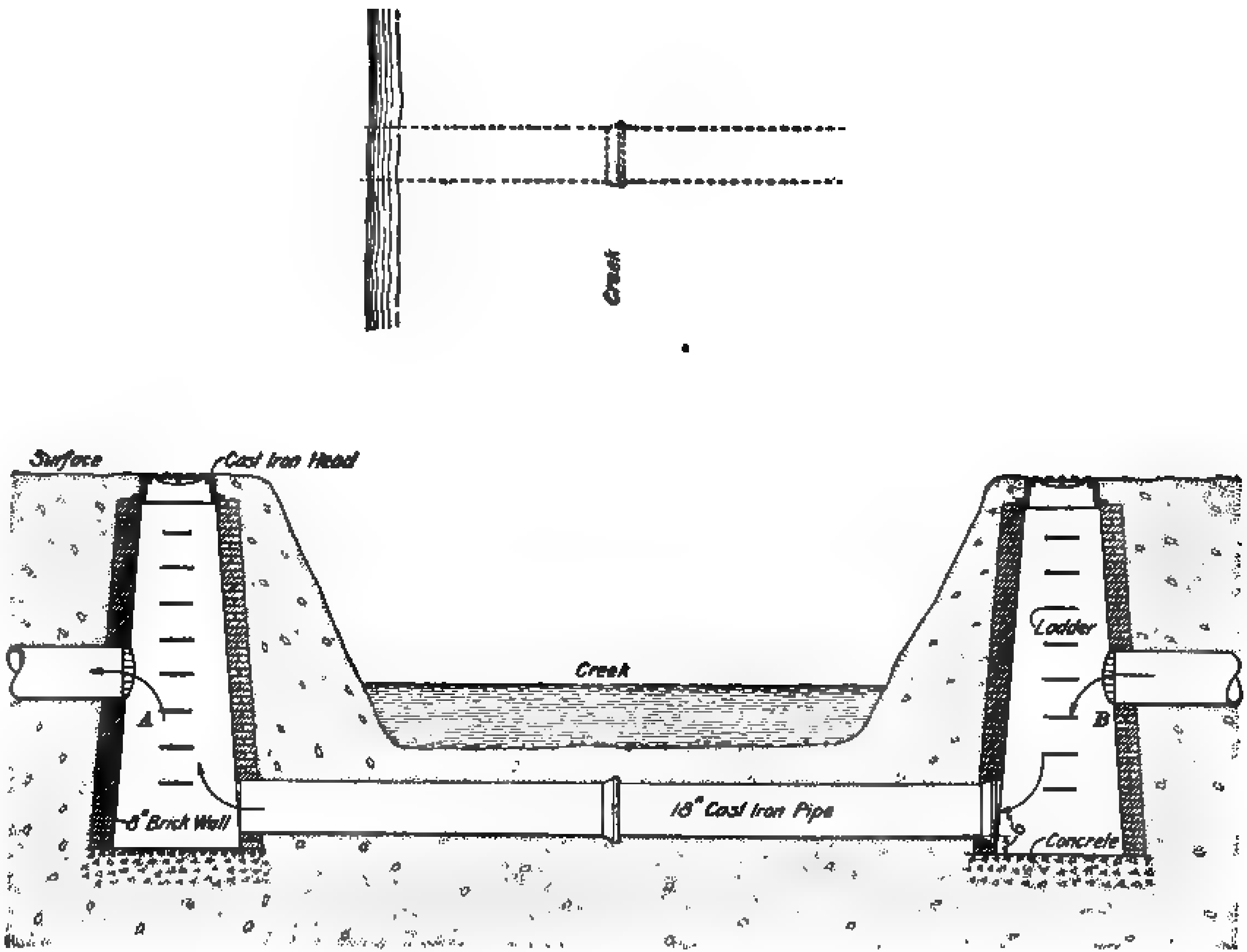

Creek
Surface
Cast Iron Head
Ladder
Creek
A
B
8" Brick Wall
18" Cast Iron Pipe
Concrete

FIG. 24

unfavorable ground, an inverted siphon is generally built of iron pipe; but if the ground is favorable and the work can be readily done in an open trench that can be kept free from water, the siphon may be built of ordinary sewer pipe. The usual method is to build at each end of the submerged pipe a manhole, at the bottom of which the siphon pipe enters, the sewer itself entering higher up on the grade that is interrupted by the stream. Fig. 24 shows a design for such an inverted siphon. Computations must be made to determine the loss of head in passing through the siphon; this loss will be about 1 foot more than that due to friction in the pipes, and the point *B* must be placed enough lower than *A* to produce a velocity of at least 3 feet per second for average flow.

FIG. 25

22. General Method of Construction for Crossings.—Stream crossings are usually made by building trestle bents in the stream and erecting an iron pipe line, which is afterwards lowered into place in such a way that the joints are not disturbed. Fig. 25 shows a crossing of this kind.

23. Special Form of Construction.—It is highly important that the velocity in an inverted siphon is not allowed to fall below a fixed minimum—about 3 feet per second—on account of possible deposits. If the pipe flowing full is designed to carry the maximum flow, even at high velocity, it may with the reduced night flow be so reduced in volume as to make deposits. In order to avoid this

difficulty, siphons are usually built with two or three small pipes so arranged that they come into service singly and in proportion to the amount of flow. Fig. 26 shows a very simple arrangement for this purpose. The circular pipe shown is the sewer entering the manhole; *A* and *B* are the two lines of pipe making the siphon. When only a moderate volume is running, the pipe *A* is the only one in action; when the flow increases, the pipe *A* is surcharged; the level of the sewage in the bottom of the manhole rises to the top of the pipe *B*, and then that pipe also comes into action.

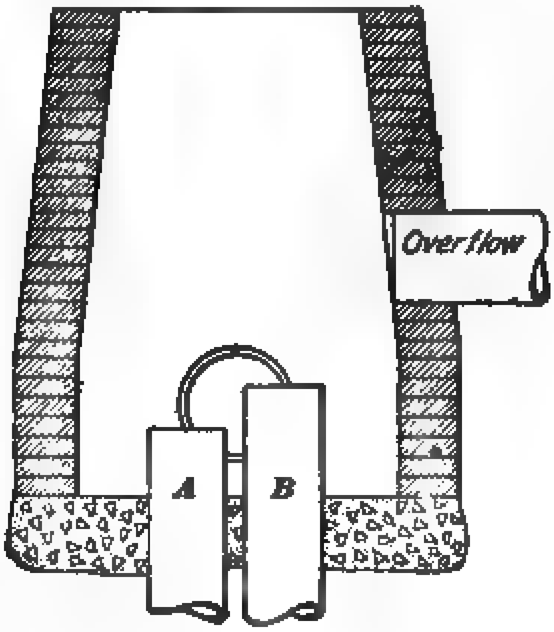


FIG. 26

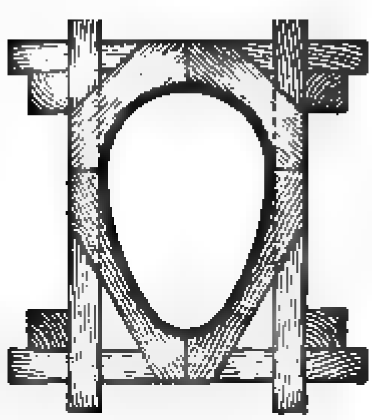


FIG. 27

24. Bridges for Sewers.—Across deep, narrow gorges, it is often economical to build a light bridge on which the

sewer pipe, carefully boxed to avoid freezing, is carried. Fig. 27, taken from "Engineering Record," shows a bridge built for this purpose. The light character of the bridge should be noted. The truss shown has a span of 25 feet, and three of these trusses, resting on pile bents in the center, carry the 32″ × 48″ egg-shaped sewer across a

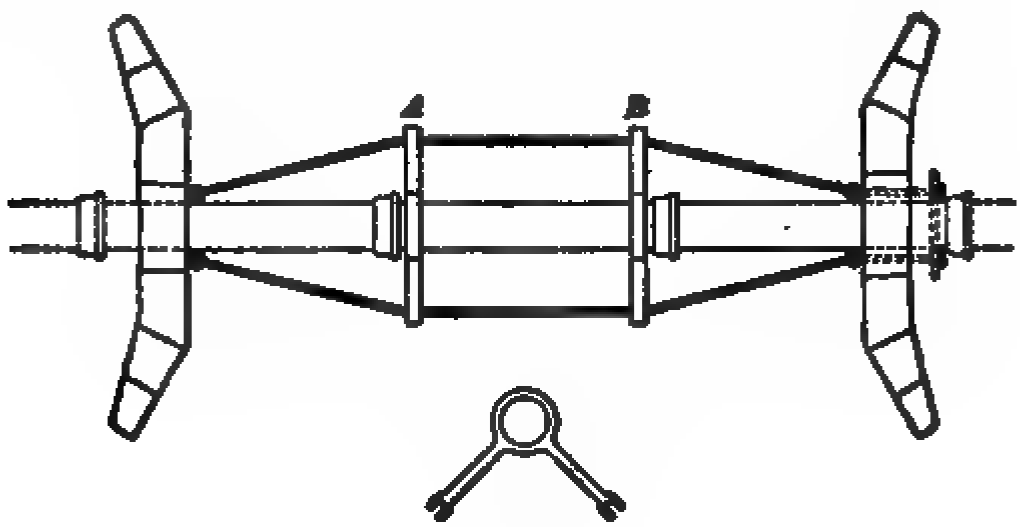
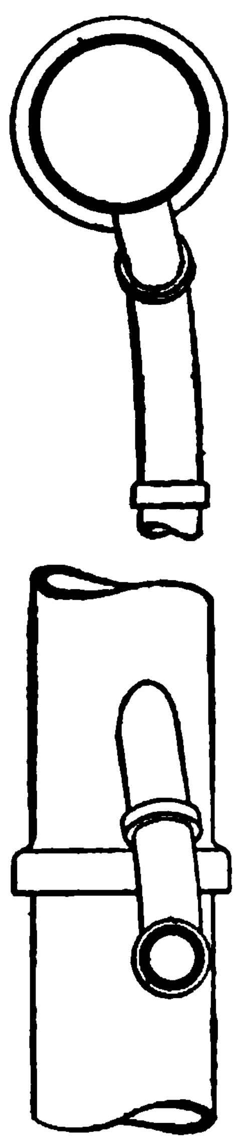


Fig. 28

stream. The sewer, up to the bridge, is brick, two courses, or 8 inches, thick; on the bridge, beginning at the abutment, the sewer is built up of two thicknesses of tongue-and-grooved yellow-pine flooring with tar paper between. The truss is a simple wooden truss, with wooden timbers for the chords and inclined members, and with vertical tie-rods. The sewer is held in place by boards sawed to fit its outside.

It is not good practice to carry the sewer on a highway bridge, because the vibration of the bridge will loosen the joints of the pipe line. Lead joints in cast-iron pipe, which would be used in such exposed places, begin to leak after but a few weeks. The screw joints of wrought-iron pipe are not affected, but such pipe is not much used, because it is not very durable.

Fig. 28 (from Folwell) shows how a sewer pipe is itself trussed and carried on the piers of a highway bridge; by this construction, vibrations, as well as the expense of a special bridge, are avoided. The span shown is about 26 feet, and the sewer pipe of cast iron is built into masonry abutments at the ends of the span. At each end of the middle length of pipe are two saddle-shaped straining pieces, under which the two tension pieces of the truss pass, their ends being anchored back into the abutments as shown.



HOUSE CONNECTIONS

25. Pipe-Sewer Y Branches for House Connections.—House connections are made to pipe sewers by means of a special detail called a **Y branch.** This detail is shown in Fig. 29. It consists essentially of a length of sewer pipe intersected by a length of smaller diameter. The angle of intersection toward the upper or socket end of the pipe is usually slightly less than 45°, so that when curves that are about one-eighth of the circle are inserted squarely in the Y openings, they lead off nearly at right angles to the sewer, as shown in the figure. Until a house connection is made to a Y branch, the end of the branch is closed by a cap or stopper, as shown in Fig. 30. The opening is first closed with a metal

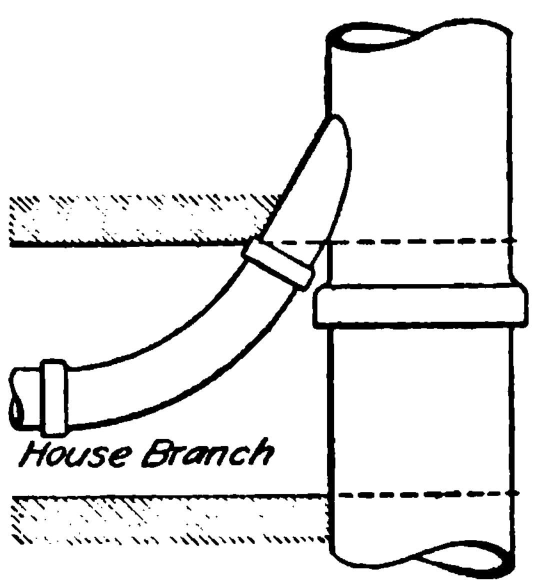


Fig. 29

or earthenware disk, and then the socket is partly filled with earth over which is plastered a coating of cement. When the Y is opened, the cement can readily be broken and the earth and disk removed without injuring the socket.

In shallow trenches, and where the plumbing is a considerable distance from the sewer, the Y branches should be tipped slightly above the horizontal; in deep trenches, and where buildings are close to the sewer, they should be set in at a steeper pitch.

FIG. 30

26. Brick-Sewer Connections.—For making the house connections to brick sewers, a piece of pipe corresponding to the Y branch of Fig. 29 is used. This piece of pipe, called a **branch** or **slant**, is built into the upper arch of a brick sewer just above the springing line, or line along which the upper arch and invert join. The form of the branch and the manner in which it is built into the side of the sewer are shown in Fig. 31.

FIG. 31

27. Method of Laying House Connections.—House branches, or house drains, are laid after the construction of the sewers, and frequently some years after the sewers are completed. It is better so to locate the Y's in the main sewer that the trench for the house branches will be approximately at right angles to the street. This trench may be perfectly straight from the sewer toward the building, and of a sufficient width next to the sewer to take in the opening of the

Y branch and leave room for the connecting curve, as shown in Fig. 29. It is generally necessary, when building these house branches, to sheet up the sides of the excavation; and this is much more easily and securely done when the trenches are laid out in a straight line.

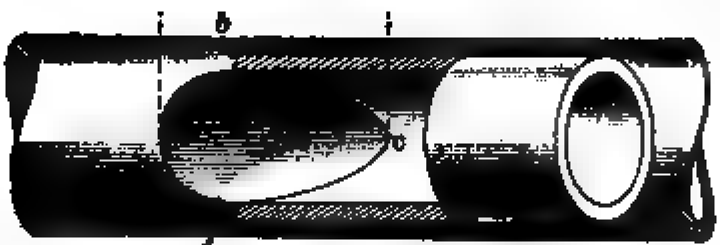
28. Size of House Branches.—House branches are usually from 4 to 6 inches in diameter, according to circumstances. In ordinary cases, 5 inches is a very good size to adopt. Soil pipes are very commonly 4 inches in diameter, and if the house drain is slightly larger than this, the danger of stoppage in it is much less.

It is advisable to adopt a uniform size for Y branches throughout the system, as otherwise much confusion may result later.

29. Grade of House Branches.—House drains require more fall than the main sewers, as the flow in them is less constant and smaller in volume. It has been ascertained by practical experience that it is not well to lay them with less fall than about 1 in 50, or about $\frac{1}{4}$ inch per foot. Where it is necessary to lay them with a less inclination than this, they should be laid with the greatest possible care and so arranged that stoppages when they occur can be conveniently removed.

BRICK-SEWER INTERSECTIONS

30. Junction Angle Greater Than 30°.—In joining two brick sewers the angle between the axes of which is 30° or more, the intersection of the two cylinders is carefully drawn out and a pattern is made for the edge or arris of intersection. In Fig. 32 are shown the plan (*a*) and elevation (*b*) of such an intersection, the front half of the branch sewer being cut away to show the arris *b c d*, for which curve a pattern is usually made. The objection to this construction is that it weakens the sewer, thus greatly impairing its capacity to resist the external pressures to which it may be subjected. As the angle of intersection diminishes, the intersection grows longer axially; the strength of the sewer



(b)

Fig. 22

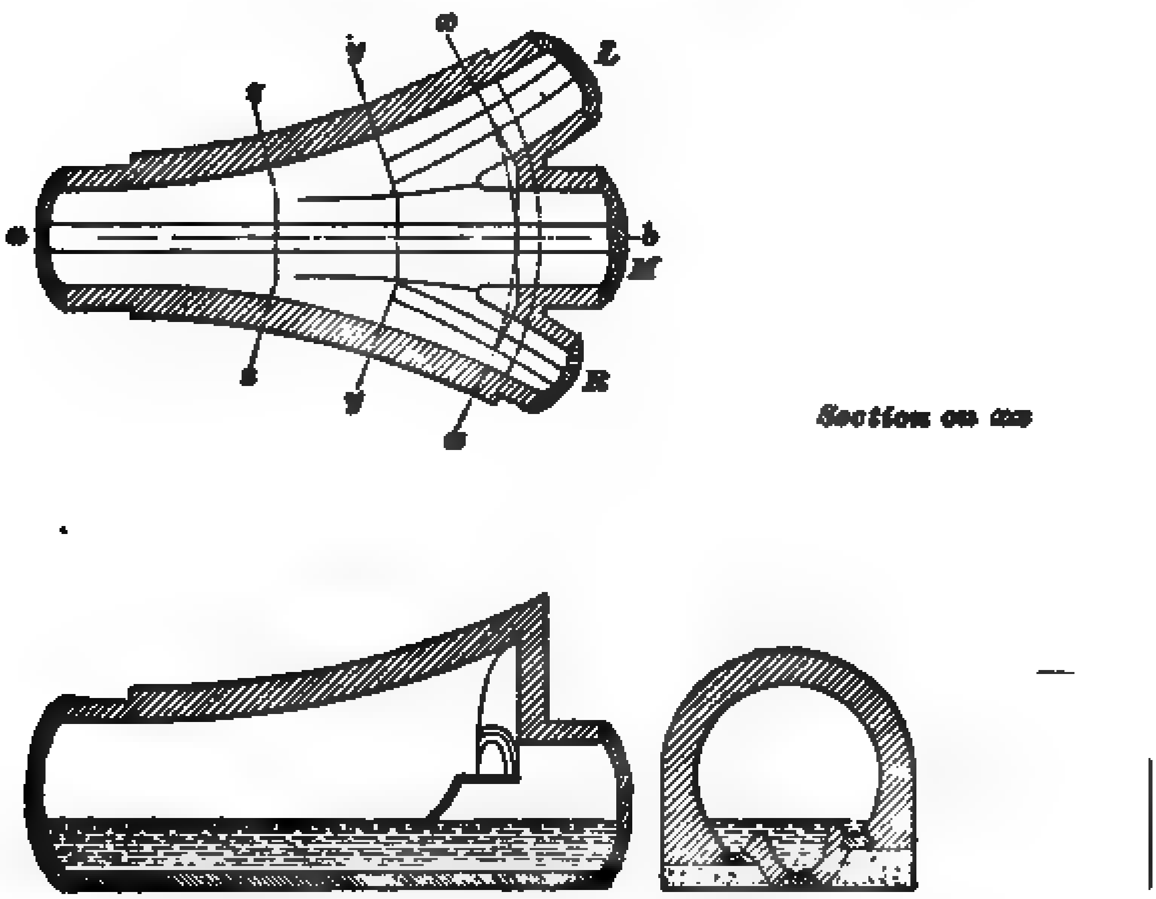


Fig. 23

is correspondingly diminished, and the sewer may even fall from its own weight as soon as the centering is withdrawn.

31. Junction Angle Less Than 30°.—When the angle is less than 30°, or where the two sewers are brought together by tangent curves, some method other than a straight intersection must be employed. Fig. 33 shows the usual construction, which is known as the **bell-mouth connection.** The main sewer M is joined by two branch sewers L and R. The arches of these three sewers all end just above the section taken at $x\,x$. At this same point, an arch is turned from the outside of L to the outside of R, covering M entirely and rising a number of feet above the latter sewer. This large arch is then gradually reduced in size, giving it the shape of a bell or trumpet, until at $z\,z$ it has been brought down to the same size as the arch of the main sewer below $z\,z$. In Fig. 33, the vertical section $a\,b$ shows how the height of the arch is reduced, as well as the span, whose reduction is shown in the plan just above.

OVERFLOWS

32. Overflows are used in the case of combined sewers carrying both storm water and house sewage, to separate the two kinds of sewage. This separation is done for two purposes; namely: (1) When the sewage is to be treated before it is discharged into a watercourse, the storm sewage, not being so foul, is taken out and discharged directly, leaving the house sewage alone to be purified. (2) When the sewage, in order to carry it to a far-away point where its discharge will not be offensive, must be pumped, the storm sewage may be taken out and discharged separately near by, thus diminishing the amount of pumping and the size of the line of outfall.

33. Diverting Weir.—The simplest method of accomplishing the foregoing object is by means of a **diverting weir**, which is a weir whose crest is at about the level of the maximum house-sewage flow. When the flow becomes heavy from rainfall, the level of flow rises and overflows the

weir, which must be long enough so that the depth of flow over it will be only a few inches. When the weir overflows, some of the house sewage goes over it mixed with the storm water; but it is assumed that the dilution is so great that the house sewage will not become a nuisance at the point where the storm water is discharged.

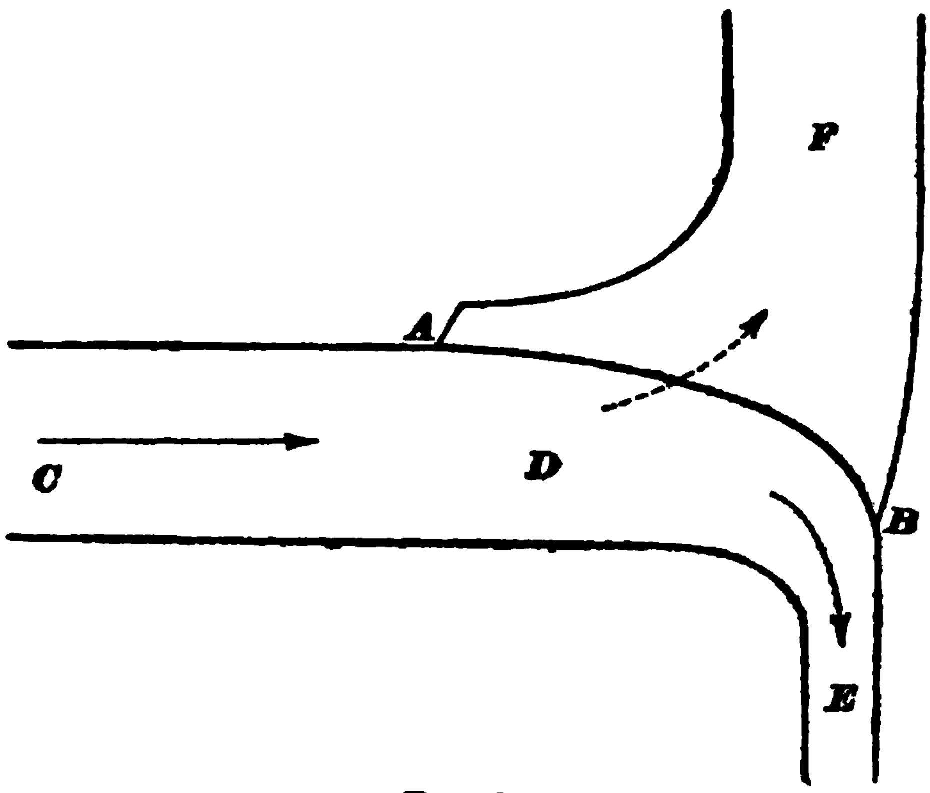


FIG. 34

Fig. 34 shows, in outline, the method of construction: CD is the combined sewer, carrying normally only house sewage, which flows into the small sewer E. In time of storm, the level of flow rises, and, after reaching the level of the weir AB, overflows and runs off through the sewer F to the point of discharge, while only a portion of the flow continues through E to be treated or pumped.

34. Leaping Weir.—A **leaping weir** for separating the house sewage from the storm flow is illustrated in Fig. 35. In dry weather, the flow in the combined sewer AB is small; the sewage falls directly into the sewer S at right angles to AB and is taken away to the pumps or to the treatment works. In time of flood, the flow shoots across the opening, as shown by the dotted arrow line, and goes off through the sewer CD to the near-by point of discharge.

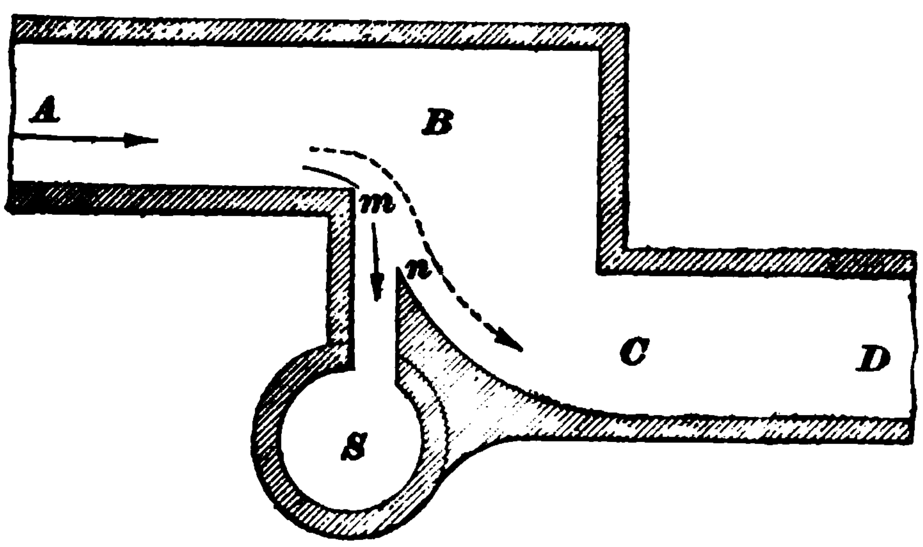


FIG. 35

It is not practicable to compute exactly the width of the opening *m n* to be left, in terms of the depth of flow in *A B* and of the difference in level between *m* and *n*; it is customary to either make the width of opening variable by providing a sliding crest at the point *n*, or to make the opening too small at first, and, after observation of several storms, cut away as much of the masonry at *n* as is necessary. Rounding the crest at *m* has the effect of carrying more water through the opening. The edge of the opening at *m* is usually made horizontal, so that the cross-section of the sewer at that point has a rectangular invert.

35. Mechanical Diverter.—A third class of overflow operates by means of a **mechanical diverter**, in which a flap valve opens and shuts to admit more or less water to the direct discharge line, as the volume of sewage in the combined sewer is greater or less. Fig. 36 shows the simplest form of diverter. The float *F* in the manhole *M* rises as the volume of sewage backs up in the manhole, and in rising opens the valve at the end of the pipe *P*. The float is adjustable, so that the discharge through *P* does not take place until the water level in the manhole reaches a certain height.

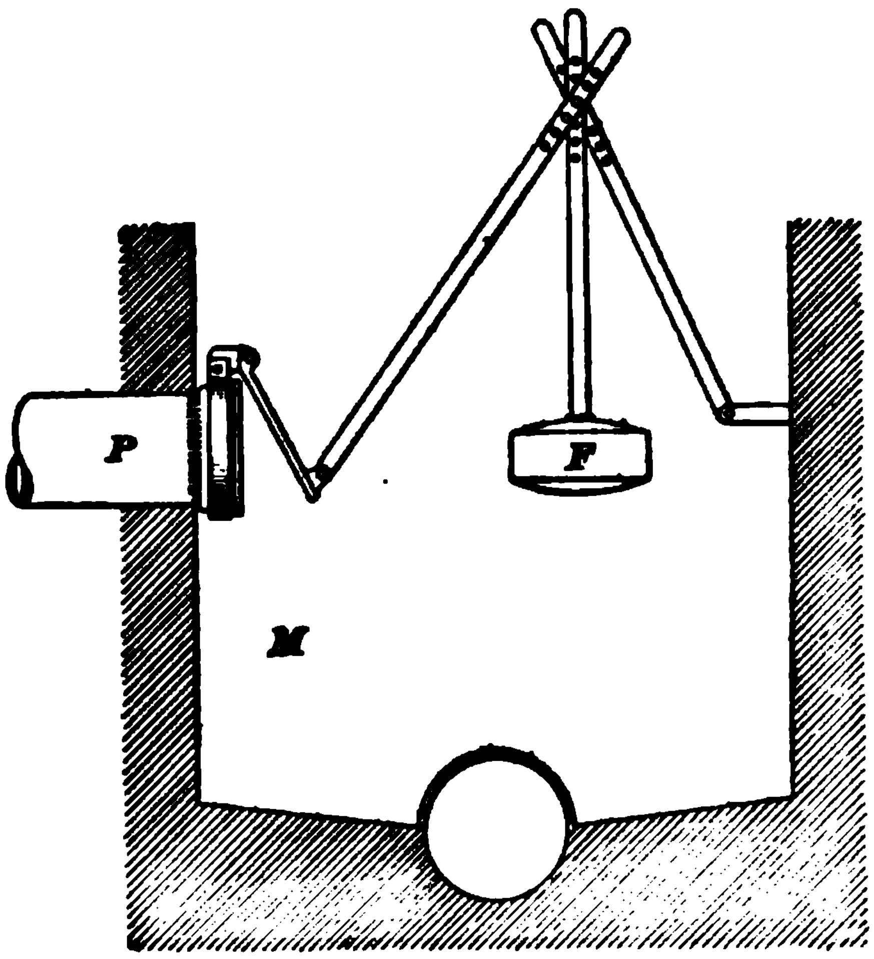


FIG. 36

OUTLETS

36. Outlets for Separate System.—When sewage is

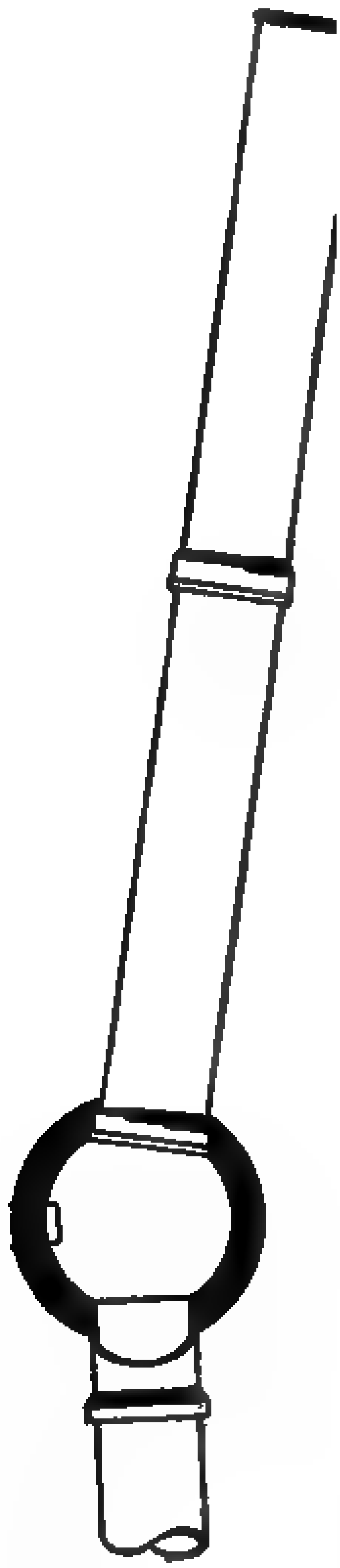


Fig. 37

discharged into a large body of water, the outlet of the sewer should be made deep enough to be covered. If the

sewers are on the separate system, a cast-iron pipe is carried out from a manhole on the bank: the pipe is buoyed up on rafts, jointed, and then sunk; or the joints may be cold-calked under water. There is no need for water-tight joints, so that pine wedges driven into each joint will answer every purpose. In rivers with a strong current, or in large lakes, where winds may stir up heavy waves, the pipe is fastened down by driving piles on each side of it at intervals of about 12 feet, and bolting timbers to the piles above and below the pipe. In a rock bottom, it will usually be sufficient to blast out a trench just deep enough to let in the pipe. Fig. 37 shows the simplest design by which the outlet can be constructed.

37. Outlets for Combined System.—Where the sewers are on the combined system and the water near the shore is shallow, the great expense of carrying the large pipe out into deep water (the distance may be several hun-

FIG. 38

dred feet at time of low water), may be avoided by carrying out a comparatively small pipe for the objectionable house sewage and letting the storm water discharge directly at the river bank. Fig. 38 shows the arrangement used for this

purpose. The large sewer ends at the bank, with or without a masonry protection wall. A small iron pipe, of a size computed to carry the house sewage, leads out from the bottom and extends into deep water. In this way, the shallow water near the shore is not polluted, as it would be with a continuous discharge of sewage.

TRENCHING FOR SEWERS

38. General Considerations.—The question of trenching is not peculiar to sewer construction, but arises in nearly all kinds of municipal improvement. The simplest method of trenching in earth is by hand labor, using pick and shovel. For deep trenches, several men may be necessary to raise the earth in stages from platform to platform. Derricks and buckets, operated by hand, by horse power, or by hoisting engines, may be employed. Mechanical excavation is often used, by which machines with three or four men dig trenches up to 20 feet in depth and 5 feet in width. Conveying machinery is sometimes installed, by which dirt excavated at one point is conveyed back to the finished sewer for back filling.

The methods followed must be adapted to the work specially in hand, and must generally be somewhat varied in detail for each particular case. Each contractor usually has his own methods, which, to some extent, he prefers to follow; this he should, in justice, be allowed to do, so long as it does not in any way interfere with the quality and prompt execution of the work. But no questionable methods, such as might result in inferior work, should be allowed under any consideration. The engineer should himself have personal charge of the construction, as matters calling for his decision will be continually arising, which, together with the locating and recording of junctions and similar work, cannot safely be entrusted to an inspector.

It is, in most cases, best to commence the construction at the lower end of the sewer and work toward the higher levels. This plan will permit the ground water to flow away through the constructed part of the sewer, will keep the trench free

from water, and is not objectionable when the volume of water is not so great nor the current so swift as to wash the cement before it has set. Pipe sewers should always be laid with the socket ends of the pipe toward the summit; when the work is begun at the lower end and proceeds upwards, the spigot of each pipe is easily inserted in the socket of the pipe previously laid.

In some cases, however, where ground water is encountered in such quantities as to render the construction difficult, the work may be prosecuted more advantageously by working downwards, or toward the outlet. This will permit the water to be drained away from the sewer into the lower levels of the trench and then pumped out. The part of the sewer under construction will be kept comparatively dry, which is in all cases desirable, and is essential when the excavation is in certain kinds of material.

39. Bracing and Sheet Piling.—Sewers are generally constructed in open trenches. In good ground, the bottom of the trench should be formed to fit the lower half of the sewer as closely as possible. The stability of all sewers depends largely on the stability of the invert, and if this is not rigidly supported at the quarters and sides so that it cannot spread when the upper arch is loaded with earth, failure may occur. In good ground, this can readily be accomplished, but in some soils it is necessary to support the invert on a foundation.

In most cases, where the depth is not great, the sides of the trench will stand without protection. In some soils, however, there will be a great tendency to caving, and it will be necessary to protect the sides of the trench by means of timberwork and braces. This is a matter to be looked after principally by the contractor, as on him falls all the risk of the undertaking. It is, nevertheless, a matter over which the engineer should keep a general oversight, as the lives of the workmen may be endangered, and the work greatly delayed, by accidents due to lack of or insufficient protection to the banks of the trench during the construction of the sewer.

The banks of sewer trenches are commonly protected by means of a temporary framing of planks and timbers, known as **sheet piling.** This consists essentially of a row of

Fig. 39

planks having their lower ends sharpened, driven vertically along each bank, and stiffened by braces extending across the trench. If the trench is deep, the planks that sustain the

bank can generally be placed horizontally with advantage for about the upper 4 feet of the trench, then driven vertically for the portion below. The construction is shown in Fig. 39.

The horizontal planks may usually be in the ordinary marketable lengths, 16 feet being a convenient length. They should be 2 inches thick. A length of about 7 or 8 feet is generally to be preferred for the vertical planks or piles, which may be about 1 inch thick, if sufficiently supported. It is more economical, however, to use 2-inch plank, which can be used two or three times, while 1-inch stuff, if driven, is entirely shattered. One row of such piling, in connection with the horizontal planking, will be sufficient for a depth

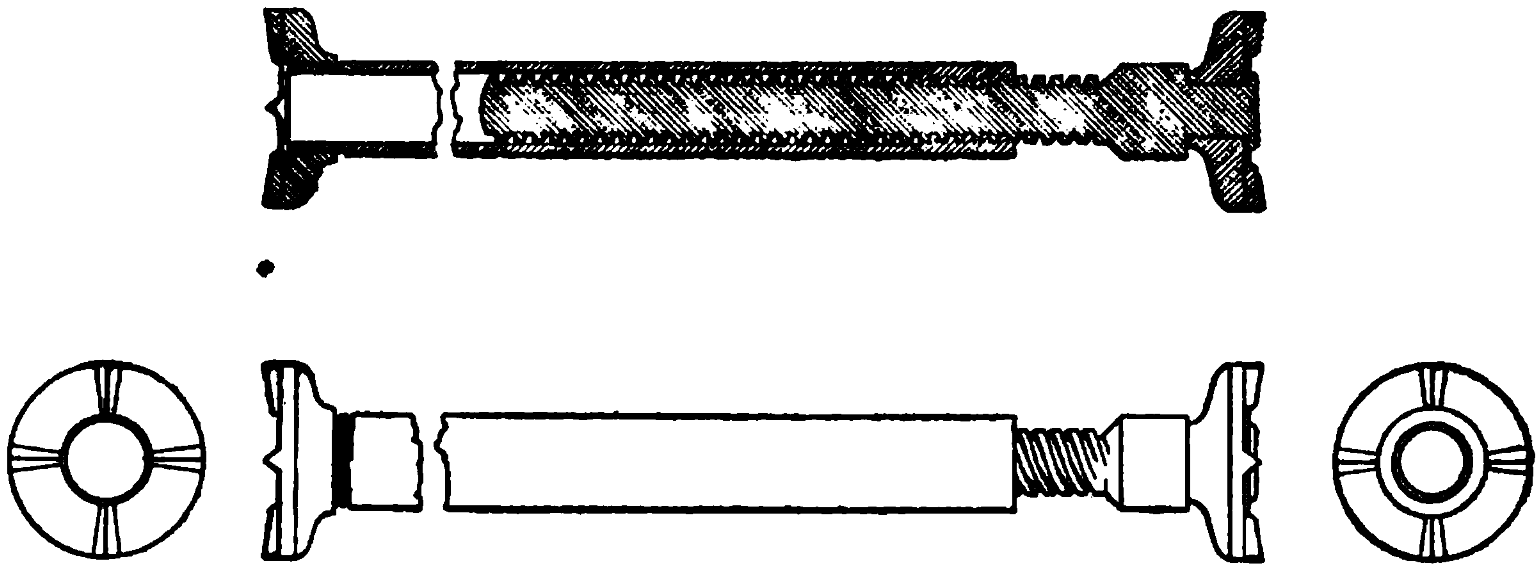


Fig. 40

of 10 to 12 feet. For greater depths, two or more rows of piling must be driven, each row being on the inner side of the next row above, as shown in Fig. 39.

The planking and piling is held in position by horizontal and vertical timbers, generally about 4 in. × 6 in. in cross-section, which are, in turn, held in place by the cross-braces. The cross-braces used are sometimes timber shores. These, however, must be cut to length and driven into place. They often become loose, and must then be wedged or replaced by longer ones. Shores used once cannot generally be used a second time. Much better cross-braces are afforded by the iron screws shown in Fig. 39 and also in detail in Fig. 40. These screws can be used any number of times, be adjusted to fit any width of trench within reasonable limits, and be quickly put in place, removed, or tightened without jarring.

40. Excavation for Pipe Sewers.—For pipe sewers, the trench can be excavated by common laborers as deep as about the center of the pipe; below this depth, it should be shaped by men trained to the work. The trench should be so formed that the pipe will be supported entirely on its cylindrical part (where it is of uniform cross-section), a recess being formed to receive the socket and cement joint for each length of pipe. This recess should be afterwards filled with well-packed sand. The pipe should be carefully laid with the socket end toward the summit; this should be done by one trained man, who should have a helper when laying large pipe. The joints should also be cemented by one man well trained to the work. The earth should then be carefully packed around the pipe before back filling; it should be packed with especial care around all **Y** branches, and about the lower half of the sewer.

41. Removal of Water From Trench.—Water is one of the most important factors in trenching. If it occurs in small quantities, a tin boat pump may be used occasionally to remove it. Better than this is a diaphragm pump, which has large valves, and pumps muddy water without difficulty. Where steam is available, steam siphons and pulsometer pumps may be rigged. The most effective method, however, is to install a system of drain tiles below the line of the sewer, either directly beneath or on one side. These tiles are usually agricultural drain tiles 4 inches in diameter, though in coarse gravel the lower end of a long line should be 6 inches in diameter. They are laid with open joints, are surrounded with gravel to keep dirt out of the joints, and discharge either into some natural watercourse, if the topography allows, or else into a small well or receptacle, such as a barrel, from which a steam pump draws day and night as long as work continues.

42. Often, where pumping is depended on for removing water, the water that collects in the trench and saturates the banks during the night is continually seeping out during the early part of the day, and, naturally, interferes with

work. It also keeps the bottom of the trench soft, and, with the treading of workmen, forms mud, which is not suitable as a foundation for the pipe. Where tile drains are laid, the trenches will be kept free from water during the whole 24 hours, the banks will gradually drain, and the bottom of the trench will be in much better condition to support the pipe.

43. Back Filling.—After the earth has been well packed about the sewer and to a point about 1 foot above it, the remainder of the back filling may be done with more or less care according to circumstances. In paved or important and much-traveled streets, it should be well compacted to the surface by ramming in layers, or the trenches may be flooded with water, which, in sandy and gravelly soils, settles them more effectually than ramming. In less important streets, particularly with sandy or gravelly soils, less care in compacting the trenches is necessary, and the earth may be rounded up over the refilled trench and left to settle by itself. When the sewer is not in a street, the back filling may be done with a scraper drawn by horses attached by means of a rope about 50 feet long.

FLUSH TANKS

44. Object of Flush Tanks.—Flush tanks are used either to collect the sewage and discharge it rapidly at intervals for the purpose of flushing the sewers, or to collect and discharge water from some public supply for the same purpose. It is only the dead ends and upper parts of sewers that require flushing, particularly where the grades in these parts of the system are slight. Farther down the line, the accumulated flow should be sufficient to keep the sewers clean.

The sewers should be flushed regularly, and automatic appliances are, therefore, most convenient, and are generally used for this purpose. Sometimes, they are omitted on steep grades, and the sewers are occasionally flushed with a hose through lamp holes placed at dead ends.

45. Kinds of Flush Tanks.—The requisites for automatic flush tanks are certainty of action, rapidity of discharge, simplicity, ease of inspection, durability, and economy both in first cost and in maintenance. Many forms of automatic flush tanks have been invented. They may generally be classified as *valve tanks*, *tilting tanks*, and *siphon tanks*.

Valve tanks discharge by means of valves generally operated by balls floating on the surface of the water.

Tilting tanks are hung on horizontal axes, and each tank is so formed that, as it fills, its center of gravity

FIG. 41

becomes changed until equilibrium is destroyed and the tank tips over and empties itself. When the tank is empty, its own weight restores it to its former position.

Siphon tanks, when they become filled to the desired point, discharge by means of siphons.

Moving mechanical parts are objectionable in a flush tank. The type of flush tank most in use is the siphon tank, and probably the best of this class is the kind known as the **deep-trap siphon**, which is illustrated in Fig. 41. It consists of two simple castings, a **U** tube, or trap, with mouthpiece,

and a cast-iron bell placed over the longer leg of the siphon and held in place by brackets cast on the trap. The action of the siphon is as follows: As the water entering the tank rises above the lower edge of the bell, it compresses the air within, the lower portion of the trap being, of course, filled with water. As the water level of the tank rises, the confined air gradually forces the water out of the long leg of the trap. Now, as the difference in the water level in the two legs equals the difference of the levels between the water in the tank and the water within the bell, it will be seen that the column of water in the short discharge leg has practically the same depth as the head of water in the tank above the level at which it stands in the bell. The two columns of water, therefore, counterbalance each other at a certain fixed depth in the tank. As soon as this depth is increased by a further supply, however small, a portion of the confined air is forced around the lower bend, and by its upward rush carries with it some of the water in the short leg, thus destroying the equilibrium.

SEWER SURVEYS AND RECORDS

46. Location of Works Previously Constructed. Before staking out the line of a sewer, it is necessary to ascertain the location of all conduits, such as gas and water pipes, that may have been previously laid in the street, in order that they may be avoided in the construction of the sewer. This is not always an easy matter. To obtain reliable information concerning the location of these previously constructed works is often very difficult, and sometimes impossible, until they are met with in the excavations for the sewer. A map of these works should be obtained when possible; otherwise, a rough map or sketch should be made from such information as is obtainable.

47. Importance of Record.—The difficulty experienced in locating and constructing the sewer among works of which no record has been made emphasizes the importance

of keeping an accurate record of the exact location of every part of the sewerage system. This record should include the position of the sewer line with reference to the street lines, the grades of the sewer, the elevations at all changes of grade, junctions and other important points, the exact location of all catch basins, flush tanks, manholes, and lamp holes, and the position of all Y branches or slants for house connections. It will also be well to keep a record of the position of all gas, water, or other underground conduits encountered in the excavation.

48. Locating the Lines.—The center line of the sewer should be carefully located on the ground with a transit, giving it such a position as to avoid all gas, water, and other pipes, so far as their positions can be ascertained. This location should start at the lower end, or outfall, and proceed upwards along the principal trunk sewer. The lines for the branch sewers should be similarly located, beginning at their junctions with the main sewer. For the purpose of checks, and to facilitate the construction of the map, the different lines should be tied together by cross-lines wherever convenient. The position, with reference to the street lines of each line thus run, and all distances along the line should be measured with a steel tape. All measurements and notes should refer to the center line of the sewer as thus run, but as this line will be within the limits of the excavation for the trench, it cannot be preserved, and should not be marked by stakes. Stakes should be set, however, on an offset line at a uniform distance to the right or left of the center line. This offset distance should generally be about 2 feet greater than one-half the width of the proposed trench. In order to avoid confusion, the offsets should, if possible, always be on the same side of the center line.

The stakes should generally be about 1 inch square, with well-squared tops, and of such lengths as to be driven flush with the surface of the street without destroying the form of their tops. Large spikes may often be conveniently used instead of stakes, where the roadway is hard. Where

extreme accuracy is required, the point of exact measurement may be indicated by a tack in the top of the stake, but so much exactness is seldom required. The stakes should be set at uniform distances along the offset line; 25 feet is a good interval. In running the center line, it will be necessary to set a few temporary stakes for transit hubs and, possibly, for the purposes of measurement, but no stakes should be left permanently on the center line, as they might lead to confusion in the construction.

49. Curves.—Where any material change occurs in the direction of the line, it should be made by means of a curve; a circular curve is generally most convenient. When the change in direction is considerable, the curve may be run in with a transit, by chord deflections, in chords of 25 feet. Intermediate points on the curve can be located by ordinates from the chord, by stretching a tape between the stations located by the transit. This is the best method to employ where the curve is of a comparatively long radius. Where the curve is of short radius, which is commonly the case, it can generally be most expeditiously located by running the tangents to an intersection; then, locating with the tape the center of the arc, and describing the arc from this center. Where the change in direction is slight, an angle may be made in the line, and the laying out of the curve omitted until the actual construction. The curve can then be readily located by offsets from the point of intersection and points on the adjacent tangents, the offsets having been previously calculated in the office.

50. Transit Notes.—Sufficient notes should be made of all the field work to preserve a record of the location of all important points, and of such other information as may be of value. In keeping the notes, the starting point of the line at the outfall should be recorded as Sta. 0. The distance between stations may be made 50 or 100 feet.

The notes should give the position of the sewer line with reference to the street lines, and the position of the offset line with reference to the sewer line. The points where

both lines of each street crossed intersect the sewer line should be noted on the latter line, and, in most cases, the offset distances to the street lines at such points should be given also. Offset distances to the street lines should be given for all points where angles occur in either the street lines or the sewer line.

All measurements should be recorded to the *sewer line*, and not to the *offset line*.

51. Reference Points.—The sewer line should be so fixed by measurements to permanent objects that its exact position may, at any subsequent time, be readily determined. This is generally best accomplished by observing the points where the line of the sewer is intersected by the prolonged lines of the sides of buildings and by other well-defined lines of permanent objects, and measuring along the prolonged line the distance from the nearest corner of each object to the center line of the sewer.

The buildings selected should be of a permanent character, such as brick buildings, and the measurements should be taken to the nearest tenth or hundredth of a foot.

52. Leveling and Level Notes.—When the sewer lines have been finally located, the levels should be taken over all the lines. This can generally be most expeditiously done by a party following the transit party. The elevations of the surface should be taken along the center line of the sewer, at intervals of 25 feet; at all street intersections; and at all points where material changes in the inclination of the surface occur. The position of the true line of the sewer can be readily obtained from the stakes set on the offset line by measuring the offset distance with a leveling rod or, after a little practice, simply by the eye. Also, if the location of about every 100-foot station is known or found, the intermediate stations can be located with sufficient accuracy by pacing.

The levels should be carefully checked on each of the bench marks established in the preliminary survey, and if these are not at convenient intervals for construction work, intermediate bench marks should be established. This will not

only check the levels, but will serve as an additional check on the bench marks.

53. Working Map and Profiles.—When the location of all the sewers has been definitely decided on, a working map of the system should be made, showing the location of all proposed main and lateral sewers and all catch basins, manholes, lamp holes, flush tanks, and other accessories. This map is for convenient reference during the construction, and need not be at all elaborate. It can often be made from the preliminary map by simply making the necessary alterations and additions in red ink. If the changes are too numerous and far-reaching to permit this, a tracing of the streets may be made from the preliminary map, and on this tracing the sewer system may be drawn as finally located.

If the preliminary plans have been carefully drawn, the profiles should answer for construction work by correcting them from time to time as the work proceeds. It is generally unsafe to make very material changes in the preliminary profiles, particularly changes that would affect the grade elevation at points where various sewers or minor systems intersect. The assumption is that the elevations of these points were determined after a consideration of all the facts, some of which may, at this stage of the work, be lost sight of.

A convenient scale for a general working map is 40 feet to the inch. A map of this kind should show, in detail, the street-car lines, if any; fence and curb lines; trees, poles, hydrants, valve boxes, and gas drips; and the houses on each side of the street. The water, gas, and other conduit lines can be drawn in as accurately as possible, the best position for the sewer indicated, and notes for the field party made out. The profile can be plotted on the same street, showing the depth of rock and the position of all pipes and conduits encountered.

54. Line and Grade Stakes.—It is important that the sewer be constructed accurately to the grade line determined on, so that the velocities in the various parts will approximate the computed velocities. Also, in order that

the sewer may be in the exact position shown on the map, it should be constructed truly along the center line as surveyed, unless deviations are made necessary by water pipes or other obstructions encountered in the excavation of the trenches, in which case complete notes of the deviations should be made.

The line of the sewer may be determined by measuring horizontally the offset distance from the stakes set on the offset line and dropping a plumb-line from the point thus determined. The position of the grade line is sometimes determined by leveling over, with an ordinary masons' level and straightedge, from the top of each stake set in the offset line, the elevation of which had been previously taken by an engineers' level, to a point about over the middle of the trench, and then measuring down a distance equal to the difference between the elevations of the stake and the grade line. This practice is not to be recommended, however, except on steep grades, where accuracy is not essential.

55. The best method of giving lines and grades is to drive stakes in pairs, one on each side of the trench, and mark on them points at equal distances above the grade line. Boards with a straightedge uppermost are then clamped or nailed on each pair even with the marks, the center line is marked on the boards, and a cord is stretched parallel with the grade. The pipes are set by measuring down from this line to the invert with a rod marked to the proper distance. The advantage of this method is that grades can be given by the engineer before the trench is entirely excavated.

56. All grade elevations should be computed and set directly from bench marks. No dependence should be placed on any estimated depth of cut that may have been computed in either the preliminary or the final survey. These estimated depths, however, serve well as an approximate check, and should be frequently referred to as the work proceeds.

When the level has been set up and the rod reading for various grade elevations and stations has been computed,

the rod should be held on the last pipe or invert completed, and the correctness of the computed reading for that point verified. This will check the instrumental work and the computations.

Particular care should be taken to keep the correct tally on the stations. It is often difficult to preserve stakes set close to the trenches. They are frequently lost by caving banks, or displaced by workmen, and a constant watch must be kept to maintain them. High stakes on which stations can be marked cannot well be kept along the side of the trench, and they are generally driven for stability flush with the surface. It is well to set witness stakes at the curb line at even 100-foot intervals, on which the station distances are marked.

57. Construction Notes.—The construction notes should contain a clear title of what and where the work is; its character; the elevation of surface and sewer grade at intervals of 25 feet; the exact location of the sewer in the street; and the exact location of all **T** branches, slants, manholes, flush tanks, and similar accessories.

If drain tiles are placed in the trenches parallel with the sewers, the notes should show their location and size, and where they terminate. All branches that are built in manholes for future use should also be noted, giving the size, elevation, and a sketch showing how they are placed.

If an artificial foundation is put in at any point, or sheeting is left in the trench, or any construction is used out of the ordinary, the notes should show at what station it is commenced and terminated; and if a price has not previously been fixed for it, and the work is being done under contract, the notes should show the extra amount of material and labor and whatever else may be required for fixing the compensation for the work.

The dates on which various parts of the work are done should be recorded. A note should be made of any unusual occurrence in connection with the work; such as accidents, refusal of the contractor to obey instructions, damage to the work by storms, etc.

These construction notes will generally be transcribed on the final records by other parties than the one that took them in the field, and it is very important that they should be complete and plain.

58. Y branches and slants that are put in for the convenience of property owners cannot be put in promiscuously, and often cannot be put in at regular distances. As the sewer is being laid, the engineer should examine the conditions of each lot, note the arrangement of the plumbing, select a convenient route for the house sewer, after consultation with the owner, and locate the Y branch or slant accordingly.

The position of all the Y branches or slants should be temporarily marked with a stake just in advance of construction; and after they have been placed, the exact station should be taken to the nearest tenth of a foot by dropping a plumb-line to the center of the opening. After some training, a person with a good eye may dispense with the use of the plumb-line in shallow trenches, if he finds by checking that he can estimate closely.

ESTIMATES OF MATERIAL AND COST

59. The engineer should prepare estimates of cost of the work, and frequently is required to make out schedules of material required. The cost of sewers varies greatly with the cost of labor, the depth and character of excavation, and the accessibility of materials. Information as to all these points should be available in the notes of the preliminary survey, as was outlined under that head.

The cost of materials can generally be readily ascertained; in it should be included the cost of freight, cartage, storage, etc. The amount of materials required can also be readily computed in ordinary construction; but, in bad ground, where artificial foundations are necessary, it cannot always be accurately determined in advance.

There is often collateral work that must be done; such as taking care of streams, sewers, and water pipes; removing

and replacing pavements, building temporary walks and driveways; and pumping water from the trenches. These items and the amount of labor required are uncertain in cost, and should be carefully considered in making estimates.

ESTIMATES FOR PIPE SEWERS

60. Amount of Pipe Sewer.—The number of linear feet of pipe sewer of various sizes can be approximately determined from the map. It is not safe, however, to use any scaled distances for estimating purposes, preliminary or otherwise; the exact distances must be taken from the notes, if they are not marked on the map. The map shows the location of the different sizes. From the distances must be deducted the space taken out by manholes and Y's, the latter being 2 feet in length. The number of Y's depends on the number of house connections, which must be determined; usually, there is one Y for each house on each side and one for each vacant lot for improved lands. Y's may be assumed on both sides at distances of 50 or 60 feet, according to the customary width of lots. A certain allowance (1 or 2 per cent., according to locality and condition of roads) should be added for breakage, but the exact lengths should be stated for contract purposes.

TABLE I

LIST PRICE OF SEWER PIPE

Diameter Inches	6	8	9	10	12	15	18	20	24	30	36
Straight pipe, price per foot	$ .30	$ .45	$ .55	$ .65	$ .85	$1.25	$1.70	$ 2.25	$ 3.25	$ 5.50	$8.85
Bends	$1.10	$1.80	$2.25	$2.75	$3.50	$4.75	$6.50	$ 7.50	$11.00	$30.00	
Y's 2 feet long	$1.35	$2.03	$2.48	$2.93	$3.83	$5.63	$7.65	$10.13	$14.63	$24.75	
Weight per foot, pounds	16	22	26	32	45	63	84	98	130	208	300

61. Cost of Pipe Sewer.—The cost of sewer pipe is listed uniformly throughout the United States by an association of factories. In 1907, the prices for straight pipe, Y's, and bends were as given in Table I.

From these prices large discounts are allowed; in 1906, the discount was about 70 per cent. for standard pipe, and about 60 per cent. for double-strength pipe. A pipe usually has to be cut at a manhole; in estimating, every such fractional length should be considered as a full length.

To the actual cost of the pipe must be added the cost of transportation and handling, and sometimes storage.

TABLE II

COST OF LAYING SEWER PIPE

(Cents per Linear Foot)

Diameter in Inches	6	8	9	10	12	15	18	20	24	30	36
Laying, jute, and calking . . .	1.5	1.6	1.8	2.0	2.3	2.6	3.0	3.7	4.5	5.5	6.8
Cement mortar .	.6	.6	.7	.8	.9	1.1	1.3	1.6	2.0	2.9	4.6

62. Cost of Laying Pipe.—The cost of laying sewer pipe is largely a matter of labor, which is modified by the character of the soil. In dry, firm soil, where the advanced laborer can trim the trench to grade, 6-inch to 12-inch pipe can be laid at a cost of labor of about 1 cent to 2 cents a foot. In wet and soft material, it will cost five or six times this amount. Average values, together with estimates for cement mortar, are given in Table II, which is taken from Folwell.

TABLE III

LENGTH OF 3-FOOT PIPE SEWER THAT CAN BE LAID WITH ONE BARREL OF CEMENT

Diameter in Inches	6	8	9	10	12	15	18	20	24
Length laid, feet . .	350	200	175	150	100	75	65	60	50

63. Amount of Cement for Laying Pipe Sewer. Table III, taken also from Folwell, gives the approximate lengths of 3-foot pipe that, under ordinary conditions, can be laid with one barrel of cement.

ESTIMATES FOR BRICK SEWERS

64. Number of Bricks in Sewers.—Table IV gives the number of bricks necessary to build brick sewers of various sizes.

TABLE IV

APPROXIMATE NUMBER OF BRICKS PER LINEAR FOOT REQUIRED IN BRICK SEWERS

Circular Sewers			Egg-Shaped Sewers		
Diameter Inches	Single Ring	Double Ring	Horizontal Diameter Inches	Single Ring	Double Ring
24	55	120	24	63	143
27	57	130			
30	62	142	30	77	170
33	68	154			
36	72	163	36	90	200
42	85	185	42	105	225
48	95	207	48	119	253
54	107	230	54	131	280
60	117	250	60	146	308
66	128	272	66	160	335
72	138	295	72	173	363
78	150	317	78	187	390
84	160	337	84	201	418
90	172	360	90	215	446
96	182	380	96	228	472

65. Amount of Mortar Necessary for Brick Sewers. Table V gives the amount of mortar necessary to lay brick sewers of various sizes; this amount includes a plaster coat on the outside $\frac{1}{4}$ inch thick. Table VI contains the amount of sand and cement necessary to make 1 cubic yard of mortar for the proportions given.

66. Cost of Brick Sewers.—The cost of brick sewers is made up of the cost of brick, cost of cement and sand,

and cost of labor. The first three items vary in different localities, and must be estimated for every particular place, not forgetting the cost of hauling, and, in the case of cement,

TABLE V

APPROXIMATE QUANTITY OF MORTAR PER LINEAR FOOT REQUIRED IN LAYING BRICK SEWERS

Circular Sewers			Egg-Shaped Sewers		
Diameter Inches	Single Ring Cubic Feet	Double Ring Cubic Feet	Vertical Diameter Inches	Single Ring Cubic Feet	Double Ring Cubic Feet
24	1.00	2.2	24	.90	1.90
27	1.04	2.3			
30	1.10	2.42	30	1.00	2.15
33	1.18	2.55			
36	1.24	2.70	36	1.15	2.40
42	1.38	2.95	42	1.27	2.60
48	1.52	3.20	48	1.40	2.80
54	1.66	3.50	54	1.50	3.05
60	1.80	3.80	60	1.60	3.30

TABLE VI

BARRELS OF PORTLAND CEMENT AND CUBIC YARDS OF SAND NECESSARY TO MAKE 1 CUBIC YARD OF MORTAR

Proportion of Cement to Sand	1 : 1	1 : 1½	1 : 2	1 : 2½	1 : 3
Barrel of 3.5 cubic feet	4.22	3.49	2.97	2.57	2.28
Barrel of 3.8 cubic feet	4.09	3.33	2.81	2.45	2.16
Barrel of 4.0 cubic feet	4.00	3.24	2.73	2.36	2.08
Cubic yards of sand	.60	.70	.80	.90	1.00

the cost of storage. The average price of brick is $12 per 1,000, with $1 additional for hauling. Cement costs about $2 a barrel, with 10 cents additional per barrel for hauling. Sand costs about $1 a yard delivered. The labor

of laying depends on the local wages of masons, and on the facilities for supplying the brick and mortar; average conditions would make brickwork cost about $4 per 1,000 bricks laid. The mortar for 1,000 bricks, at the prices just given for sand and cement, costs about $3.50, making the total cost of 1,000 bricks in place in a sewer as follows:

1,000 brick	$12.00
Hauling	1.00
Laying	4.00
Mortar	3.50
Total	$20.50

It should be understood that these are average figures, which may not apply even approximately under special conditions.

OTHER ITEMS OF COST

67. Cost of Earthwork.—The cost of trenching is the most uncertain part in estimating the cost of a sewer. The character of the soil so affects the cost of digging that the same-sized trench in one soil may cost ten times what it does in another. The cost of sheeting or bracing is sometimes necessary and sometimes not. Where the trenching is more than about 6 feet deep, a laborer on the bank must be provided to throw back the dirt from the edge, after it has been thrown out of the trench. When trenches are over 8 feet deep, platforms must be built in the trench, and there must be a laborer at the bottom throwing dirt on the platform, and one throwing it out from the platform. If a laborer can handle dirt at a cost of 20 cents per cubic yard, it follows that, while that figure may be used in trenches up to 6 feet deep, from 6 to 8 feet it will cost 40 cents per cubic yard, and from 8 to 12 feet, 60 cents per cubic yard, on account of the necessity of having men on the platforms.

The widths of trench are estimated as follows: for 6-inch to 15-inch pipe, 3 feet; for 18-inch to 24-inch pipe, 4 feet. This is only approximate, since a trench 6 feet deep for a 6-inch pipe may be only 2 feet wide, and a 25-foot trench

for a 24-inch pipe, if sheeting is necessary, must be made about 6 feet wide.

68. Cost of Sheeting.—Records kept on sewer work show that in small trenches, 8 to 16 feet deep, close sheeting costs about 20 cents per linear foot, exclusive of the timber, which may be used about three times under average conditions; so that the cost of timber needed may be divided by 3. Trenches 12 feet wide and 35 feet deep cost for labor alone $1.77 per linear foot; and trenches 13 feet wide and 45 feet deep, $3 per linear foot. The lumber needed in the latter case costs about $4.80, one-third of which is used three times. This makes a total of $4.60 per linear foot, or about 21 cents per cubic yard of excavation.

69. Cost of Pumping.—In wet trenches, the cost of pumping is an important item, the more so because it is usually underestimated. Table VII, taken from Gillette, is a concise statement of the probable cost. Slightly wet trenches are those where one diaphragm pump would be used. Quite wet trenches are those where a steam pump is installed, with a special drain pipe leading to sump wells 500 feet apart.

TABLE VII

COST OF PUMPING FROM TRENCHES

Depth of Trench. Feet	5	10	15	20	25
Slightly wet, cost per foot . .	$ .06	$ .07	$ .10	$ .12	$ .18
Quite wet, cost per foot . . .	.71	.73	.76	1.04	1.27

Other records show that, where a trench 6 to 8 feet deep is in wet gravel, and the bottom of the trench is about 1 foot below ground-water level, the cost of pumping is about 15 cents per linear foot.

70. Cost of Rock Excavation.—Rock excavation is usually estimated for a certain width of trench, irrespectively of the actual width excavated. That is, for sewers up to 18 inches in diameter, the yardage is usually estimated and

paid for as if the trench were 3 feet wide; for sewers 18 to 24 inches in diameter, as if 4 feet wide, allowing about 9 inches, roughly, on each side of the outside of the sewer. The cost is estimated per cubic yard, and runs from $1 per cubic yard for soft sandstone, to $3 per cubic yard for hard granite or trap. For limestone, $1.75 per cubic yard is a fair average.

71. Back Filling.—The cost of back filling depends on the thoroughness with which tamping or puddling is done. Simply casting the dirt back in a trench of moderate dimensions, where the dirt can be thrown in without walking, costs about 10 cents per cubic yard. If the earth is tamped even perfunctorily, the cost rises to from 15 to 20 cents. In city streets, where the tamping is thoroughly done, or where the laborers walk with their shovelfuls only for a few steps, the cost rises to 30 or 40 cents per cubic yard.

72. Material and Cost of Manholes.—The number of brick required for manholes is shown in Table VIII, the bottom not being included. If of concrete, a load of gravel and a barrel of cement will make the bottom. If of brick, 200 bricks with one-half barrel of cement and two wheelbarrow loads of sand for mortar must be added. The cost of the brickwork has already been explained.

TABLE VIII

APPROXIMATE NUMBER OF BRICKS REQUIRED IN MANHOLES

(8-Inch Walls)

Depth Feet	6	7	8	9	10	11	12	13	14	15	16
Number of bricks	1,100	1,250	1,400	1,550	1,700	1,850	2,000	2,200	2,350	2,500	2,650

It has already been explained that the weight of the cast-iron frames and covers is about 350 pounds, costing from $1\frac{1}{2}$ to 3 cents per pound; this amount must be added to the cost of the bottom and walls just given.

73. Other Appurtenances.—The cost of the special designs and arrangements used in connection with sewers is often difficult to estimate. Cast iron can be estimated by the pound, although special castings in small quantities often cost 4 or 5 cents per pound, besides the cost of the wooden patterns.

Lumber for foundations, sheeting, etc. may be estimated at the local price delivered with from $5 to $10 per 1,000 feet added for labor in placing. In the form of wooden pipe, the labor is difficult to estimate, being proportionately larger for small sizes, but $15 per 1,000 feet should build any rough-lumber structure.

Machine work—needed on gates, valves, etc.—adds greatly to the cost of such accessories, and should be carefully considered in making estimates.

Lamp holes, consisting of a little concrete, extra lengths of pipe, and a T and a casting at the top, may be estimated in parts.

Flush tanks cost, in place, from $40 to $85.

Catch basins may also be estimated in parts, the cost of which has been already considered.

PURIFICATION OF WATER
(PART 1)

QUALITY OF WATER

IMPURITIES IN WATER

1. Pollution of Rivers and Lakes.—When it is considered that drinking water is the source of many diseases, the necessity for securing a good wholesome supply of water and safeguarding it in every possible way to maintain its purity cannot be overestimated. Numerous instances can be cited of epidemics of cholera and typhoid fever caused by an infected water supply.

2. Lake and river waters are seldom free from sewage pollution. Cities, villages, and country dwellings along the shores usually empty their sewage into the waters, and the surface washings from the adjacent territory are added to the filth of the already polluted stream or lake. Formerly, it was believed that running water purified itself, and that a few miles below a source of pollution the water was fit for use again; investigation, however, has disproved this hypothesis. It is now known that running water effects only a partial clarification by dilution, and that within 20 miles of the source of contamination slow-running river water should not be used, while with swift-current rivers the minimum limit should be 30 miles. Even then, waters from an acknowledged contaminated source should not be used without previous filtration.

Lake waters are clarified much more rapidly than river waters. In lakes free from currents, the range of pollution is usually confined to the vicinity of the sewer outfall. **Sedimentation** is the chief factor in clearing lake water of sewage. The large particles fall to the bottom, and in falling carry with them smaller particles with which they come in contact, thus effecting a still further clarification.

3. Many waters that are not badly polluted with sewage carry so much silt and loam in suspension that they present a muddy appearance. Such waters are known as **turbid** waters, and in their **raw** (that is, unclarified) condition are unfit for most industrial purposes, besides being objectionable for domestic use.

4. **Turbidity of Rivers.**—**Turbidity** is caused by the presence of large quantities of suspended matter in water. Sometimes, it is caused by countless microscopic organisms or minute forms of animal and vegetable life growing in the water; this form of turbidity, however, is comparatively rare, and, owing to its unimportance, may be ignored. Turbidity is occasionally caused by erosion of the bed of the stream, but generally the clay and silt carried in suspension are washed by the rain from the adjacent soil. The matter carried in suspension by turbid streams is principally *inorganic matter;* that is to say, mineral matter.

Small streams are most turbid at times of heavy rains; the turbid water quickly runs away, however, and the water flowing in the stream between heavy rains is comparatively clear. In large rivers, the duration of the turbid period is longer, because the turbid waters from different tributaries arrive at a given point in the river at different times. The turbidity of a river is naturally greater during the flood period, when the water of the tributaries is the most turbid, and the river remains turbid as long as flood water from any tributary is passing. Some rivers have such large watersheds and so many tributaries that their waters are turbid during a large part of the year.

5. The turbidity of a stream depends largely on the geological character of its watershed. River beds and watersheds of rocks that are too hard to be much affected by the elements, as also sandy and very firm soils, yield but little turbidity to water falling on them. Cultivated land yields more turbid water than meadow or wood land. The waters of New England, New York, Michigan, and certain other sections of the United States have usually but little turbidity. The waters of many southern and western American rivers are exceedingly turbid.

6. Measure of Turbidity.—Turbidity is expressed in parts per million, by weight, of suspended matter; or, better still, as equivalent to a certain number of parts per million of suspended matter of a specified grade of fineness. The standard of comparison for turbidity is a water that contains one hundred parts of silica per million in such a state of fineness that a bright platinum wire 1 millimeter in diameter can just be seen when the center of the wire is 100 millimeters below the surface of the water, and the eye of the observer is 1.2 meters above the wire. The turbidity of such water is assumed to be 100. In comparing with this standard, the observation is to be made in the middle of the day, in the open air, but not in sunlight, and the water is to be contained in a vessel large enough so that the sides do not shut out the light sufficiently to influence the results. This is the **silica standard** adopted by the United States Geological Survey.*

Waters more turbid than the standard are diluted with clear water until the mixture is of the same degree of turbidity as the standard. Each equal volume of clear water added to turbid water reduces the turbidity by just one-half. Thus, if a water whose turbidity is 200 is mixed with an equal volume of clear water, the mixture will show a turbidity of 100.

7. The turbidity of water is computed as follows: Let v denote the original volume of the water examined; v_1, the volume of the mixture after sufficient clear water has been

*Recommended by Allen Hazen and George C. Whipple.

added to bring the mixture to the standard turbidity; and t, the turbidity of the water examined. Then, since the total amount of suspended matter is the same in the volume v as in the volume v_1, the amounts per unit of volume, and therefore the turbidities of the original water and the mixture, are inversely proportional to the volumes v and v_1; hence, since the turbidity of the mixture is 100, we have the proportion,

$$v : v_1 = 100 : t$$

from which
$$t = \frac{100\, v_1}{v}.$$

EXAMPLE.—What is the amount of turbidity, according to the silica standard, of water that contains the equivalent of 100 parts of silica per million after having been diluted by 2.5 volumes of clear water?

SOLUTION.—If the original volume of water is taken as unity, we have $v = 1$, $v_1 = 2.5 + 1 = 3.5$. Substituting these values in the formula,

$$t = \frac{100 \times 3.5}{1} = 350. \text{ Ans.}$$

8. Field Measurement of Turbidity.—Turbidity is determined in the field by noting the depth at which a platinum wire 1 millimeter in diameter can be seen beneath the surface of the water, a depth of 100 millimeters being taken as indicating 100° of turbidity, the same as just described for determining the standard. If a platinum wire is not available, a bright new pin can be substituted. The platinum wire is inserted in a rod from which it projects at right angles, and the rod is graduated in such a manner that, when the end bearing the platinum wire is immersed in the water, the depth at which the wire can be seen, as indicated by the graduation mark on the rod at the surface of the water, will correspond to the turbidity of the water as determined by the method described in the preceding article, the graduation mark 100 being at a distance of 100 millimeters from the center of the platinum wire. Table I, from the standards of the United States Geological Survey, gives the depths below the surface of turbid waters at which a standard platinum wire can be seen.

TABLE I

TURBIDITY OF WATER

Turbidity	Depth of Wire	
	Millimeters	Inches
10	794	31.26
15	551	21.69
20	426	16.77
30	296	11.65
50	187	7.36
75	130	5.12
100	100	3.94
150	72.0	2.83
200	57.4	2.26
300	43.2	1.70
500	30.9	1.22
1,000	20.9	.82

9. Permissible Turbidity.—For water supply, there is a limit of turbidity beyond which waters are wholly unfit for use without previous clarification; 10 is the allowable limit of turbidity for a good water supply. In water with a turbidity of 10, a platinum wire of standard size can be seen at a depth of 31.3 inches. If such a water is placed in an ordinary pressed-glass tumbler, the turbidity will be visible to a person that looks at the water closely, but will not be noticed by a casual observer, nor will it be likely to cause general complaint.

While 10 may be considered, under average conditions, the permissible limit of turbidity for a good water supply, many cities are supplied with water having much higher turbidities, in some cases the turbidity reaching the comparatively high rate of 100. Turbidities as high as 1,000 have been observed in streams from which water supplies are obtained, but such conditions are abnormal and seldom last for a great length of time.

10. The turbidity of a river at a given place varies greatly from day to day, and even from hour to hour. Fig. 1 shows the fluctuations in turbidity of the Alleghany River at Pittsburg, Pennsylvania, during 1 year.* In this figure, the amount of turbidity is represented by the height of the black portion above the straight lower edge. It will be noticed that there are many and decided fluctuations in the turbidity between the first of January and the last of August, the highest line showing exceedingly turbid water for a very short time (1 day) in March. During September and the

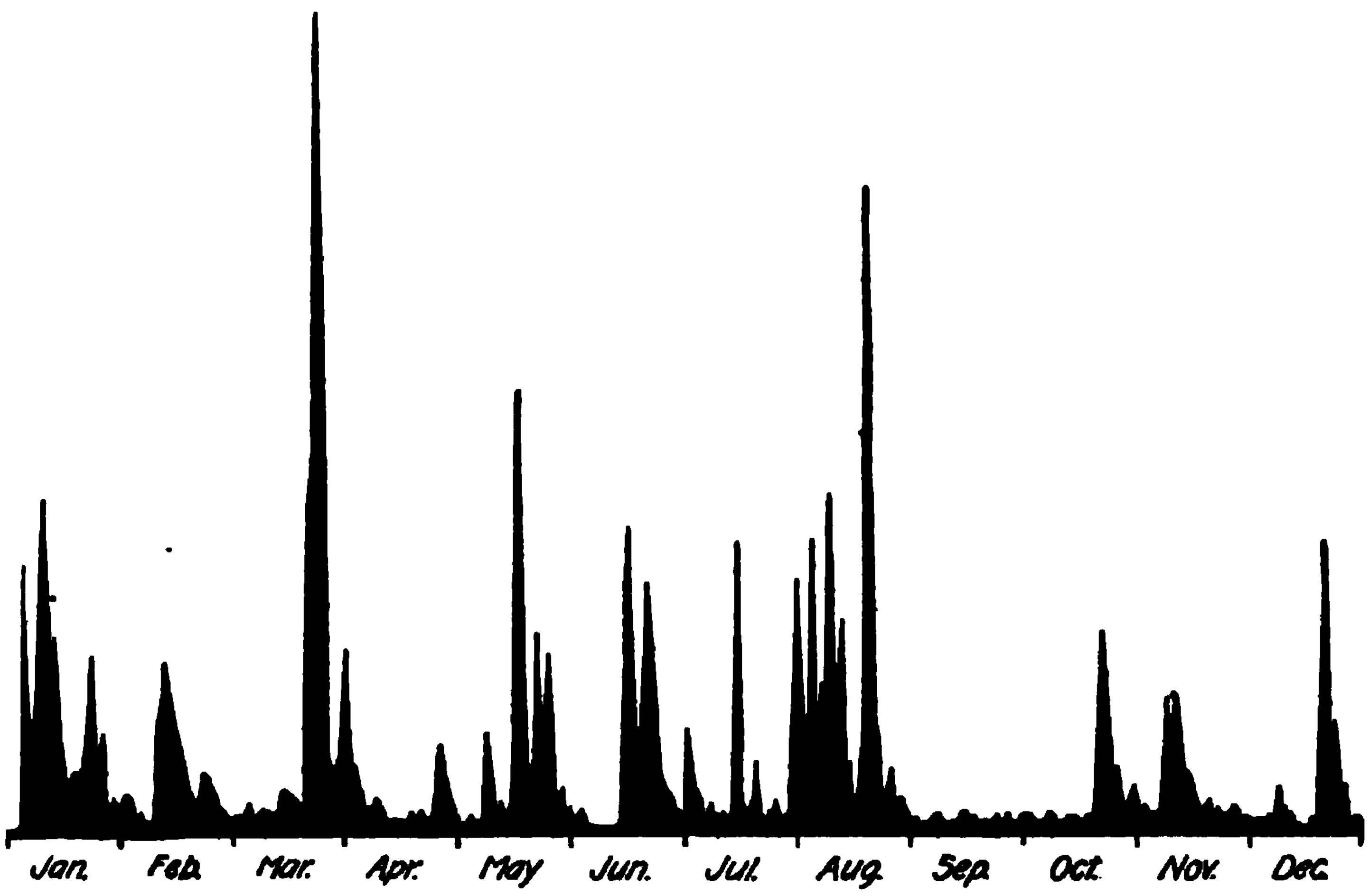


FIG. 1

first half of October, the water was but slightly turbid, and the amount of turbidity remained comparatively uniform.

11. Color.—Color, in water, is generally derived from the solution of matter from dried leaves, peat, logs, and other vegetable substances. Sometimes, it is due to iron or manganese combined with organic matter. Colored water is yellow when seen in small depths, and dull red when seen in great depths. The coloring matter is similar in character to, if not identical with, the coloring matter of tea, and colored waters have the appearance of very weak tea. It seldom

* From "Filtration of Public Water Supply" by Allen Hazen.

happens that a water is both highly turbid and highly colored. The upper Mississippi is a colored water. The Missouri is a turbid water. After mixing, the water at St. Louis is both turbid and colored, but the turbidity predominates.

Colored water mixed with turbid water soon loses its color. The loss of color is probably due, in a great measure, to the presence of alumina in the clay that causes turbidity. The alumina acts as a chemical reagent, either simply decolorizing the water, or forming with the coloring matter a precipitate that falls to the bottom. Perhaps it acts slightly in both manners. At all events, if colored water is mixed with turbid water, and the mixture is subsequently clarified, the process of clarification will show that the water has also been decolorized.

Coloring matter in water is mostly in solution. In this respect, color differs from turbidity. In turbidity, the muddy appearance of the water is caused by particles held in suspension.

12. Measurement of Color.—Color is measured by comparing the colored water with a solution of platinum and cobalt of known strength. The result is stated in parts per million, by weight, of metallic platinum required to produce an equal color. This is known as the **platinum-cobalt method** of measuring color. The standard solution, which has a color of 500, is made by dissolving in 100 cubic centimeters of concentrated hydrochloric acid, 1.246 grams of potassium platinic* chloride, containing .5 gram of platinum, and 1 gram of crystallized cobalt chloride containing .25 gram of cobalt, then adding distilled water to make 1 liter in volume. By diluting this solution, standards of comparison are prepared having values of 0, 5, 10, 15, 20, 25, 30, 35, 40, 50, 60, and 70, which values correspond to the parts per million of metallic platinum contained in the various solutions. For instance, if 1 liter of water is added to a standard solution having a color of 500, the resultant mixture will

*Not potassium platinous chloride; this salt has a reddish color, while the platinic salt used in the solution has a yellow color.

have a color of 250. If 9 liters of water is added to a standard solution, the mixture will have a color of 500 ÷ 10, or 50.

13. Standard-color solutions are kept in clear glass jars of uniform size. In order to determine the color of any water under examination, a similar jar is filled with the water to the same depth; this is compared with the standard solutions and rated according to the number of the solution with which its color corresponds.

Such standards afford a definite basis of comparison for determining the color of water, but they are suitable only for laboratory use, since it is impracticable to carry them to the field for making field observations. For field measurements, it is customary to use standard disks of amber-colored glass whose colors have been rated to correspond with the solutions of the platinum-cobalt standard. Water that has a color greater than 500 can be diluted with clear water, as in the case of turbidity, to bring the color down to or below the standard.

14. Permissible Color in Water.—The coloring matter usually contained in water is not injurious to the human system, but is less pleasing to the sight than is a clear water, and is unfit for many industrial purposes. There is no recognized standard of color permissible in a water supply, although the better practice limits the amount of color to 40. Above this value, water would be considered high-colored and objectionable to most consumers. Water with a color less than 20 would seldom be complained of. Such water, when viewed in a porcelain-lined bathtub or in a decanter placed on a white cloth, has a noticeable color; but, when viewed through an ordinary pressed-glass tumbler held in the hand, the color is not noticeable. In the absence of a permissible color standard, it may be assumed that a color value of 30 would be acceptable to most communities, and that water colored to above 40 would be objectionable.

15. Bacterial Pollution.—**Bacteria**, as defined by the Standard Dictionary, are extremely minute cells, without chlorophyl (chlorophyl is the green coloring matter of plants),

consisting of single rod-shaped or corkscrew-like cells or aggregates of cells . . . and occur as refuse eaters or parasites. Although most of them are harmless, others cause various diseases. They are widely distributed in air, water, the alimentary canals of animals, etc., and enter into all putrefactive processes.

16. Bacteria are found in all water supplies. The number of bacteria, however, varies greatly, according to the source of the supply. Well and spring waters are more free from bacteria than are lake, river, or other surface waters. Water drawn from lakes or rivers into which sewage is discharged contains more bacteria than water obtained from a source free from sewage pollution. Furthermore, water obtained from a sewage-polluted source is more likely to be contaminated by disease germs than water derived from an unpolluted source. The disease contamination of waters is caused by the excreta of patients suffering from bacterial diseases. The excreta are discharged into the house-drainage system and find their way thence through the city sewers to the outfall in the lake or river where the sewage is discharged. If the water of that lake or river is used, without previous filtration, for drinking, the persons that drink it are liable to contract the disease caused by the bacteria.

17. Tastes and Odors.—Tastes and odors in water are usually due to sewage or some other form of decomposed organic matter; sometimes, they are caused by the waste products of gasworks and other manufacturing establishments. Water having tastes and odors due to such causes is usually so polluted as to be absolutely unfit for domestic purposes. The tastes and odors that are least injurious to health, but, nevertheless, owing to their nauseating features, must most frequently be removed, are those arising from the death and decomposition of micro-organisms; that is to say, of minute plants and animals that have grown and died in the water. The organisms producing these tastes and odors are very much larger than, and entirely different in character from, bacteria. They require a food supply, and grow

most freely in reservoirs that have not been adequately cleaned and have rich soil and other organic matters on the sides and bottoms; they grow also in reservoirs into which sewage or other polluting substances capable of sustaining their lives are discharged. The organisms producing tastes and odors do not grow in very turbid waters, but they often grow in colored waters. A mixture of ground and surface water furnishes the best medium for the growth of water micro-organisms. The surface water furnishes the seed, and the ground water provides the mineral food necessary to life. It is, therefore, advisable, when water is obtained from both ground and surface sources, to store the waters from the different sources in separate reservoirs and not mix them until they are about to be delivered to the distributing mains.

18. Iron-Impregnated Waters.—Waters containing iron are called **chalybeate waters.** They are usually derived from ground sources. Sands, gravels, and rocks almost always contain iron as ferric oxide. The water that percolates through the soil often has carbonic acid and organic matter in solution. Through the action of these substances, the ferric oxide is transformed into carbonate of iron, which dissolves in, and is therefore carried by, the water. This action, however, does not take place in waters containing free oxygen. Occasionally, iron is found in water in the form of iron sulphate.

Water containing iron, either as a carbonate or as a sulphate, in amounts greater than .5 part per million, is unfit for most domestic and industrial purposes, besides having a disagreeable inky taste. Iron is objectionable in the manufacture of paper on account of the rust spots it causes. In dyeing and bleaching establishments, it modifies some of the colors and stains linens and other white fabrics a rusty brown. In laundries, it causes rust spots on the goods. In distributing systems where the velocity is low, iron deposits in the pipes; when it is stirred up by an increase in the velocity or by the opening of a hydrant, it is carried to the

house faucet, where the water appears as a rusty-brown stream.

The most objectionable feature to iron in water, from the householder's point of view, and the one most likely to cause complaint, is the growth in the pipes of an organism called **crenothrix.** While living, crenothrix cause no trouble, but as soon as they die, their decayed remains give to the water disagreeable tastes and odors.

HARDNESS

19. Causes of Hardness.—The property of water known as **hardness** is due to the presence, in solution, of bicarbonates, sulphates, chlorides, or nitrates of lime or magnesia. Water not containing these salts is said to be **soft.** Sometimes, hardness is due to a combination of two or more forms of lime and magnesia. Thus, hardness may be caused by bicarbonates and sulphates of lime, by sulphates and bicarbonates of magnesia, or by sulphates of lime and bicarbonates of magnesia. Hardness that is due to bicarbonates of lime or of magnesia is known as **temporary hardness,** and can be removed by boiling the water for a sufficient length of time in an open vessel: the prolonged boiling drives off the combined carbonic acid, leaving a solid residue that sinks to the bottom of the vessel. **Permanent hardness** is due to solutions of sulphates, chlorides, or nitrates of lime or magnesia, and can be removed only by chemical treatment.

20. Measurement of Hardness.—Hardness may be measured either in **degrees Clark** or in **degrees Frankland.** One Clark degree of hardness is equivalent to 1 grain of carbonate of lime in 1 imperial gallon of water. In the Frankland scale, 1 degree of hardness is equivalent to 1 grain (or part) of carbonate of lime in 100,000 grains (or parts) of water. Hardness expressed in degrees Clark can be converted into parts per 100,000 by dividing by .7.

21. Determination of Hardness.—The greatest objection to hardness is that it diminishes very considerably

the capacity of the water to form a lather with soap. The salts imparting hardness to the water form with the soap a solid combination that goes to the bottom as a precipitate, and not until after a great deal of soap has been thus "destroyed" is it possible to obtain a lather. Hardness is, accordingly, determined or measured by the **soap test**; that is, by the amount of soap of standard strength that will be destroyed in softening a certain quantity of the water to be tested. The quantity of water used in a soap test depends on whether the hardness is being determined in grains per gallon or in parts per 100,000. When the determination is in grains per gallon, 70 cubic centimeters of water is used. The reason for this is that 70 cubic centimeters contains just as many milligrams as a gallon contains grains, and may therefore be regarded as a gallon with milligrams representing grains. When hardness is being determined in parts per 100,000, 100 cubic centimeters of water is used, and each milligram of water corresponds to 1 grain or part per 100,000.

22. The soap test is applied in the following manner: 70 cubic centimeters (or 100 cubic centimeters, as the case may be) of the water to be tested is placed in a clean glass bottle large enough to hold two or three times that quantity. A clear solution of soap of standard strength is then gradually added to the water, and the mixture briskly shaken. At first, a slight lather may form on the surface of the water, but, if the water is still hard, the lather will break up and disappear. More soap should then be added in small quantities, and the bottle shaken after each addition until a lather is formed that is sufficiently permanent to last for several minutes. The number of cubic centimeters of soap solution added to the water, less one, indicates the hardness of the water in degrees. One degree is deducted because even distilled water requires a slight quantity of soap to make it lather.

23. Standard Soap Solution.—A standard soap solution is a mixture of soap and water of such a strength

that 1 cubic centimeter of the solution will exactly neutralize 1 milligram of dissolved carbonate of lime. It is made by mixing ½ ounce of finely shredded Castile or mottled soap with 2 pints of methylated spirits, and 1 pint of distilled water. The mixture should not be warmed, but should be permitted to stand for a few hours, shaking occasionally, then passed through a filter of blotting paper. Before using the solution, it should be tested by means of water of known hardness, and, in case it is too strong, it should be diluted with spirits and water until the required strength is obtained.

24. Objections to Hard Water.—Hardness in a water supply is objectionable for many reasons. In laundry work, a large percentage of the soap used with hard water is destroyed by the lime or magnesia contained in the water. It is estimated that, in addition to the soap actually used for washing, the lime salts in 1,000 gallons of water will destroy 1.7 pounds of soap for each degree of hardness. The lime salts combine with the soap fats and form a greasy, useless, and insoluble curd that adheres to the fabrics so tenaciously that it is with difficulty washed out. In wool scouring and cleaning or other washing processes in the manufacture of textiles, hard water is objectionable for the same reason that makes it unsuitable for laundry purposes. In dyeing work, it has the additional effect of modifying some colors.

25. Hard water is particularly undesirable for use in steam boilers. When heated to about the boiling point, much of the lime and magnesia in solution is precipitated in the form of an insoluble compound that adheres to the shell of the boiler. This layer of lime not only reduces considerably the heating capacity of the boiler, but is likely to cause the overheating and consequent damage of some of the plates. Lime deposits on boiler tubes often cause them to burn out or leak.

26. Lime is a prolific cause of trouble in range waterbacks and water heaters. In localities where the water supply is very hard, waterbacks and water heaters become completely choked with lime, and require cleaning sometimes as often as once a month.

27. Permissible Hardness.—Hardness is not a dangerous, but only a disagreeable, quality of water, and therefore there is no exact line to be drawn between the amount of hardness permissible in water and that which is not permissible. There are many drinking waters in constant use with a hardness of 10 to 20 parts per 100,000. Such waters are disagreeable for use in bathing and washing, so that, if possible, it is better to provide a water whose hardness is not over 10 parts per 100,000.

EXAMINATION OF WATER

GENERAL CONSIDERATIONS

28. Methods of Testing Water.—From whatever source water is derived, it should never be adopted for domestic supply without first subjecting it to a chemical and bacteriological examination. When an examination of water is to be made, it is better for the analyzing chemist or his assistant to collect the samples himself, and every opportunity should be given him to study the conditions surrounding the source of supply, as this will enable him to interpret more intelligently the analysis on which his report is based.

In making an examination of water, four kinds of tests are employed; namely, *physical*, *chemical*, *bacteriological*, and *microscopic*. Not all these tests are needed in order to pronounce judgment on the quality of the water, but each test is of great value for interpreting the results of the others, and all four ought to be employed in any matter of importance.

29. Physical Tests.—Tests for *color*, *turbidity*, *odor*, *taste*, and *temperature* are physical tests. No water is perfect with regard to all these features, and only those waters that are actually repulsive would be rejected on the result of the physical tests alone.

30. Interpretation of Analyses.—The methods employed in the analysis of water are often exceedingly

delicate and exact. Organic matter in certain forms can be measured in as small a quantity as 1 part in 100,000,000. The engineer, however, is not, as a rule, concerned with these methods themselves, but with the results. The analyses are usually made by a chemist, who reports the results to the engineer, and the latter must be able to interpret those results; that is, to decide from them whether the water is suitable or not.

31. Collecting Samples.—The collecting of samples of water for analysis should be done very carefully. The following directions have been given by a chemist recognized as a high authority on water analysis.*

"Large glass-stoppered bottles are best for sampling, but as they are seldom at hand, a new 2-gallon demijohn should be employed, fitted with a new soft cork. Be careful to notice that no packing straw or other foreign substance yet remains in the demijohn, and thoroughly rinse it with the water to be sampled. Do not attempt to scour the interior of the neck by rubbing with either fingers or cloth. After thorough rinsing, fill the vessel to overflowing, so as to displace the air, and then completely empty it.

"If the water is to be taken from a tap, let enough run to waste to empty the local lateral before sampling; if from a pump, pump enough to empty all the pump connections; if from a stream or lake, take a sample some distance from the shore, and plunge the sampling vessel 1½ feet below the surface during filling, so as to avoid surface scum.

"In every case, fill the demijohn nearly full, leaving but a small space to allow for possible expansion, and cork securely. Under no circumstances place sealing wax on the cork, but tie a piece of cloth firmly over the neck to hold the cork in place. The ends of the string may be afterwards sealed if necessary.

"Bear in mind, throughout, that water analysis deals with materials present in very minute quantities, and that the least carelessness in collecting the sample may vitiate the

*"Examination of Water" by Wm. P. Mason.

results. Give the data of taking the sample, and as full a description as possible of the soil through which the water flows, together with the immediate source of possible contamination."

It may be added to the above that every possible item of information bearing on the history of the water, from the source to the point of sampling, should also be collected for the use of the analyst in making his report, and if possible he should make a personal investigation of the territory. This should always be done in case of any doubt as to interpreting the results of the analysis.

32. Dangerous Elements.—Of all kinds of contamination to which water is exposed, by far the most dangerous is animal refuse, and of this the worst is that from human beings. Animal refuse as a source of contamination is broadly known as sewage. This is the most dangerous kind of pollution; it is the source of many diseases, and the sanitarian should, therefore, pay particular attention to it.

When it is known that a certain water is free from sewage, it is comparatively simple to ascertain whether or not it has other harmful qualities. When water is contaminated with sewage, it requires no further test to prove that, without artificial purification, it is unfit for human use. The difficulty intervenes when uncertainty exists; recourse is then had to chemistry and bacteriology as aids to the investigations, but even with these aids the results do not, of themselves, usually afford any conclusive evidence, but merely assist in tracing the history of the water.

33. General Sanitary Examination.—Perhaps as important an examination as can be made of the quality of a water consists in a careful inspection of the watershed. Pollution of water supplies may or may not appear in a chemical or bacteriological examination of the water. But if on inspection a sewer is found emptying into a stream a short distance above the proposed point of supply, the analyses are unnecessary to prove the unfitness of the water, and no report from a chemist, however reputable he may be,

will serve to disprove the evident pollution. A thorough examination would involve following up both banks of the stream to be drawn on and noting the proximity to the stream and its tributaries of houses, barns, and pig pens. There should also be noted the location and drainage of factories, the density of population on the watershed, and every other factor that may cause pollution. If there is a choice between several sources of supply, that one should be chosen which seems the freest from human drainage, and has the greatest opportunity for the contamination, if any, to be deposited in pools and quiet reaches above the point of supply. If there is only one source available, then all objectionable places should be removed or improved over the entire watershed, particularly within 1,000 feet of the streams. All sewage should be purified before its effluent is discharged into the main stream or any of its tributaries. Drains leading from barns and pig pens should not be discharged directly into the water, but should be carried out to land where the polluted water will undergo at least a partial purification before getting into the stream. Privies should not be built near or over the watercourse, and household discharges or slops should not be thrown on ground that slopes directly to the stream.

CHEMICAL ANALYSIS

34. Value of Chemical Analysis.—The determination by chemical analysis of the matter contained in water is of itself of little value, because, while all the constituents of a given sample may be accurately determined in this way, it is impossible to tell, from the analysis alone, whether they are present in the water under innocent or harmful conditions. This statement is particularly true when applied to the determination of disease germs. The analysis merely indicates certain probabilities, but no final conclusions can be drawn from them without information of a different character.

35. Metallic Substances.—Most metals are poisonous, and when present in sufficient quantity make the water unfit

for a domestic supply. It is not often that an ordinary analysis shows the presence of any of the metallic substances, such as iron, copper, lead, or arsenic. When there is a probability that any of these metals exists in water, special tests must be made to find out the kind and quantity of metals present. The ordinary chemical examination determines only the presence in certain quantities of inorganic or metallic substances, but does not determine the kind of metal.

36. Chlorine.—The presence of chlorine in water is readily and accurately determined. It is one of the elements always tested for, always found, and always reported. In itself, chlorine is not dangerous, since it exists in the form of common salt, which, in the amount indicated by any water analysis, is not injurious nor otherwise objectionable. Its determination is of value in judging a water because, in urine, which is a part of all sewage, there is a certain amount of salt, usually in the proportion of about 500 parts of chlorine to 100,000 parts of water. Ordinary surface water contains 1 or 2 parts, and the inference is usually justifiable that, if a water is found to contain 5 or more parts of chlorine per 100,000, the presence of which cannot be accounted for in any other way, the excess of chlorine is due to sewage pollution. In the vicinity of the ocean, well waters will have a normal percentage of chlorine higher than the water from wells further inland. For example, at the end of Cape Cod, all the good wells show about 2 parts of chlorine. Along the shore of the mainland, the amount is but .6 part, and in the western part of the state, .1 part is the normal amount. Three parts per 100,000 in the western part of the state would therefore be a suspicious amount, while at the end of the Cape, 5 or 6 parts would be necessary to arouse suspicion.

37. Experiments are often made with salt to determine the possibility of pollution. A well near a barn yard, for example, may be suspected, and it is necessary to ascertain whether the water is pure. If a chlorine test is made, and

the amount is found to be less than 1 part, the well is probably not polluted by barn-yard drainage. But if the water contains 6 parts of chlorine, it should be regarded with suspicion. If a bushel of salt is thrown into the barn yard, and, after a rain or at the end of 24 hours, the well water is tested and found to contain a much larger amount of chlorine than it usually contains, the conclusion is inevitable that the salt has gone through the ground to the well, and that the drainage from the yard has followed the same path.

38. Organic Matter.—The amount and character of the organic matter present in water is indicated by the amount of nitrogen. Not that nitrogen and organic matter are synonymous, but rather that the amount of nitrogen furnishes the most practical method of estimating the amount of organic matter. Chemical analysis has not succeeded in devising any method of testing for organic matter itself. It has no way of knowing whether the nitrogen it finds belongs to animal or to vegetable matter. It cannot tell whether it comes from dead animals or from decaying leaves, whether it is harmless or not. The amount of organic matter is only assumed from the amount of nitrogen present, and this is scarcely more than an indication. Organic substances are unstable compounds of carbon, nitrogen, hydrogen, and oxygen, all of which are found, in different proportions, in all organic matter. In the presence of light and air, the original proportions change; gases are given off, carrying hydrogen and carbon, and the nitrogen remains, partly in the original organic form, and partly combined with hydrogen, forming ammonia. Ammonia itself is not stable, and in the presence of air changes to nitrites and finally to nitrates, the latter being the stable form of the final salt.

39. Albuminoid ammonia represents the nitrogen present in the original organic matter before any change of condition occurs. It may represent all the organic matter if it is entirely fresh. Decaying vegetation, dissolved by water in passing through swamps, making the water brown or yellow, produces a large amount of albuminoid

ammonia. Such waters, unless otherwise polluted, will show little or no free ammonia, but perhaps .02 part per 100,000 of albuminoid ammonia. Normal waters unpolluted by sewage or by decaying vegetation usually contain less than .01 part, though the indication of bad water is not conclusive unless the amount increases to .03 part or over. This last amount is sufficient to condemn the water, unless, from the position and surroundings, it is certain that the ammonia comes from vegetation, and that there is no possibility of any human contamination.

40. Free Ammonia.—Free ammonia is driven off from water directly by heat, and is thus collected and measured. Its formation marks the first step in the decomposition of organic matter. It is most characteristic of sewage, and always forms more readily with animal matter than with vegetable. Good waters and waters in their natural condition do not contain free ammonia. Samples of water, therefore, that day after day show free ammonia indicate contamination at some near-by point. Many analyses made of the surface waters of Massachusetts for 2 years, including waters used for drinking purposes, show an average amount of free ammonia of .004 part per 100,000. The amount varies, however, very much during the year. During the summer months, when vegetation is active, absorbing the ammonia as fast as it is formed, the average was .0019 part; but in winter, when opposite conditions prevail, the amount rose to .0067.

A polluted water analyzed in the summer may not indicate pollution, since the ammonia is absorbed by vegetation so rapidly as to make the analysis entirely misleading. The winter analysis, taken in connection with the chlorine, would show the character of the water, but a summer analysis might easily lead to serious error. The average amount of free ammonia in good water varies from .001 in summer to .005 at the beginning of winter. **Free ammonia in greater quantities is an indication of human contamination.**

41. Nitrites.—Nitrogen in the form of nitrites is present in normal waters only in exceedingly small amounts. Frequently, a chemist reports its presence as a **trace**; that is, a quantity too small to be measured. Nitrites are transitory compounds, soon converted into nitrates. If the organic matter is fresh, the nitrate stage has not been reached. If the contamination is old, the nitrate stage has been reached, so that it is only in the case of a continuous pollution by organic matter that nitrites in any measurable quantities are found. Even then, the amounts are small. Any amount greater than .001 part probably means pollution, and makes the water suspicious. If free ammonia is above .01 and nitrites above .001, there probably has been recent pollution; and, if the chlorine is high, say 10 parts, sewage pollution can be at once recognized. If the chlorine is low, this fact removes the human element, and indicates that the pollution must come from another source, probably from some farmyard or manured field.

42. Nitrates.—The presence of nitrates means that the organic matter has been present, but has passed through its round of changes and become inorganic. The natural inference is that the pollution occurred some time before the analysis, and that, the organic matter having disappeared, the danger also is over. This may or may not be the case. High chlorine and high nitrates certainly show the complete oxidation of what must have once been organic animal matter. The danger of polluted water, however, does not lie wholly in the presence of organic matter, but to a great extent in the presence of bacteria, which are always found wherever organic matter exists. Even if the organic matter has passed away, it is not certain that the bacteria have also disappeared. The presence of nitrates, therefore, makes it necessary to find the cause of the previous contamination, or to examine the water for bacteria.

In good average surface waters, the nitrates are about the same in amount as the albuminoid ammonia—about .01 part per 100,000—although very pure waters may have much less.

Badly polluted waters may have but little more, if the pollution is very recent and the nitrogen is all in the earlier stages.

43. Ground water, which percolates through the ground, is likely to be high in nitrates derived from organic matter in the soil, or from some other sources, the process of percolation allowing sufficient time for the reduction of organic substances to the nitrate form. In thickly settled districts, wells—even without direct contamination and with the water perfectly safe—will always show a relatively high amount of both nitrates and chlorine. With chlorine, indeed, it is known that the normal amount in water on any area varies directly with the density of population in that area.

44. Oxygen-Consuming Capacity.—The oxygen-consuming capacity test, made by heating the water to be examined, in the presence of permanganate of potash, is made as a rapid substitution for or addition to the other tests for the presence of organic matter. The permanganate, in the presence of an acid, is broken up; its oxygen is liberated and combines with the organic matter present in the water. Much of this matter contains carbon, which combines with oxygen to form carbon dioxide. The nitrogen also combines with some of the oxygen, and the amount of permanganate used measures the amount of organic matter. This is a convenient test for gauging the efficiency of a filter or of any other purification process. Ordinary surface waters will decompose from 5 to 7 parts, although spring waters will not require more than about 2 parts of permanganate per 100,000.

45. Remarks on Chemical Analysis.—From what has been stated, it will be seen that it is impossible to determine the quality of water from a single analysis. If chlorine, ammonia, and nitrates were characteristic of the decomposition of organic matter only, water analysis would become an exact method of determining the degree of pollution. But these substances are not dangerous in themselves, and may be present in appreciable quantities in a potable water.

Briefly stated, a chemical analysis of a sample of water giving a result high in chlorine, ammonia, or nitrites, indicates sewage contamination. Local examinations of the watershed must determine whether this is possible. If it is found that sewage or other organic wastes are the cause of the presence of chlorine, ammonia, or nitrites, the source of supply should be condemned, or the water treated by filtration.

BACTERIOLOGICAL AND MICROSCOPIC EXAMINATIONS

46. Object and Interpretation of Bacteriological Analysis.—Of late years, the bacteriological examination of water has come into great prominence. A bacteriological examination usually consists in ascertaining the number of bacteria per cubic centimeter contained in a given sample of water.

Bacteria are so minute that they cannot be counted individually. The method pursued is to multiply them by cultivation in a *culture jelly*, made of gelatine, beef extract, and albumen. When the water under examination is mixed with this jelly, each bacterium in the water throws off a cluster of its offspring, forming a *colony*, which grows to be sufficiently large to be seen by the naked eye. Each colony is then counted as one original bacterium. Bacteria are the germs of zymotic or infectious diseases, and, therefore, all bacteria are looked on with suspicion, although many varieties are harmless. Hence, from the number of bacteria in a given water, the relative purity of the water is inferred. The bacteria of certain zymotic diseases have been identified. However, since so many dangerous bacteria are unrecognized, the presence of a large number of bacteria constitutes a possible danger that the water may communicate disease, and the greater the number of bacteria, the greater the menace.

47. Microscopic Examination.—No examination of water with reference to its quality is complete without a

microscopic examination. This is not expected to determine the presence of bacteria, but rather the presence of larger but still minute forms of vegetable life. A great deal of information as to the character of the water may thus be obtained. The chemical examination may show high ammonia, but will not indicate the source. If the observer finds cotton or woolen fibers, starch grains or undigested muscular tissue, or eggs of parasites that live on men and other animals, he can infer sewage pollution in the water, and should condemn the use of such water for a public supply. Water may contain both vegetable and animal organisms. The latter are not common except in water so foul and stagnant that its use would be condemned merely from its physical appearance. Certain forms of vegetation often found in clear bright water may, however, if developed in sufficient quantity, change an otherwise good supply to one extremely objectionable. These forms of vegetation are simple filaments, green, brown, or colorless, which, under certain conditions of light, food, and air, may multiply with great rapidity and give off to the water certain penetrating oils, which produce disagreeable odors. Their presence indicates an abundance of organic material in the water. The study of these **algæ**, as they are called, is the province of the specialist, although many waterworks superintendents who have found the water under their care infested have learned by experience the peculiar habits of some of the species. It is usually only after the excessive development of these algaic growths that they have been studied, while a microscopic examination of the water, if regularly made, might have found the algæ at the beginning, recognized their dangerous rate of growth, and led to the selection of another source of supply.

SEDIMENTATION AND COAGULATION

48. When a water supply is obtained from a river that carries a large amount of clay or loam in suspension, settling basins are usually provided to effect a partial **clarification** by allowing the water to stand until part of the suspended matter settles to the bottom. This settling of suspended matter is called *sedimentation*. Twenty-four hours has been found a sufficient period of time to allow for sedimentation. Most of the particles that will settle at all will do so in that time, while a longer storage might allow the growth of algæ and in other ways cause the water to deteriorate rather than improve in quality. Three settling basins are generally built, each with a storage capacity for a supply of water for 24 hours. One of the settling basins can then be filled while the suspended matter in another is settling and water from the third is being delivered to filter beds, as will be explained farther on.

The matter removed in 24 hours by sedimentation averages about 60 per cent. of the particles held in suspension. The heavier particles are drawn to the bottom by the force of gravity, and in settling carry with them such light particles as they come in contact with.

DESIGN OF SETTLING BASINS

49. Size, Number, and Type of Settling Basins. In designing settling basins, it is necessary first to determine the number, type, and size of basins to be used. These considerations are so closely related that a decision regarding one affects the others. Formerly, it was the practice to make settling basins large enough to hold from 3 to 18 days' supply of water. This practice is still adhered to in Great Britain; but in Continental Europe and in the United States,

provision for 24 hours has been accepted as the best practice, although special conditions may make it advisable to provide greater storage capacity. For instance, if, for several days at a time, during flood periods, the water of a river is so turbid as to be wholly unfit for use, it will be necessary to provide settling basins or reservoirs of sufficient capacity to tide over the greatest probable period of turbidity. It should be noted, however, that in such cases the extra capacity is for storage purposes only, and does not materially affect the sedimentation.

50. The number of settling basins required depends on the type of basin to be used. In the earlier American practice, **fill-and-draw** settling basins were used, in which the raw water was allowed to stand at rest during the period of sedimentation, and was then drawn off. When this type of basin is used, three should be constructed, so that one can be filled while one of the others is emptying and the other is being cleaned, repaired, or in reserve.

The latest American practice favors **continuous-flow** sedimentation. With this method, only two settling basins are required, one of which is in use while the other is being cleaned, repaired, or in reserve.

51. Continuous-Flow Basins.—A settling basin of the continuous-flow type is shown in Fig. 2. A rectangular reservoir is divided into passages by baffled walls *a*, *a* that confine the water to a particular course. The bottom of the basin slopes toward a channel *b* that extends diagonally across the bottom of the basin from near the inlet to a large mud-valve, through which the sediment is washed out. Hydrants *h* are provided at suitable intervals along the sides of the basin to facilitate washing out the mud. The raw-water inlet is connected to the settling basin near the bottom. This pipe should be enlarged where it enters the basin, to reduce the velocity of the flow and prevent the formation of eddies, which would interfere with the process of sedimentation.

The outlet should be located at least 3 or 4 feet below the surface of the water, to exclude floating objects, and avoid

the danger of clogging with ice. It may, if desired, be placed near the bottom, so that the basin can be used on the fill-and-draw plan.

52. **Fill-and-Draw Basins.**—A fill-and-draw settling basin differs from a continuous-flow basin in that it has no

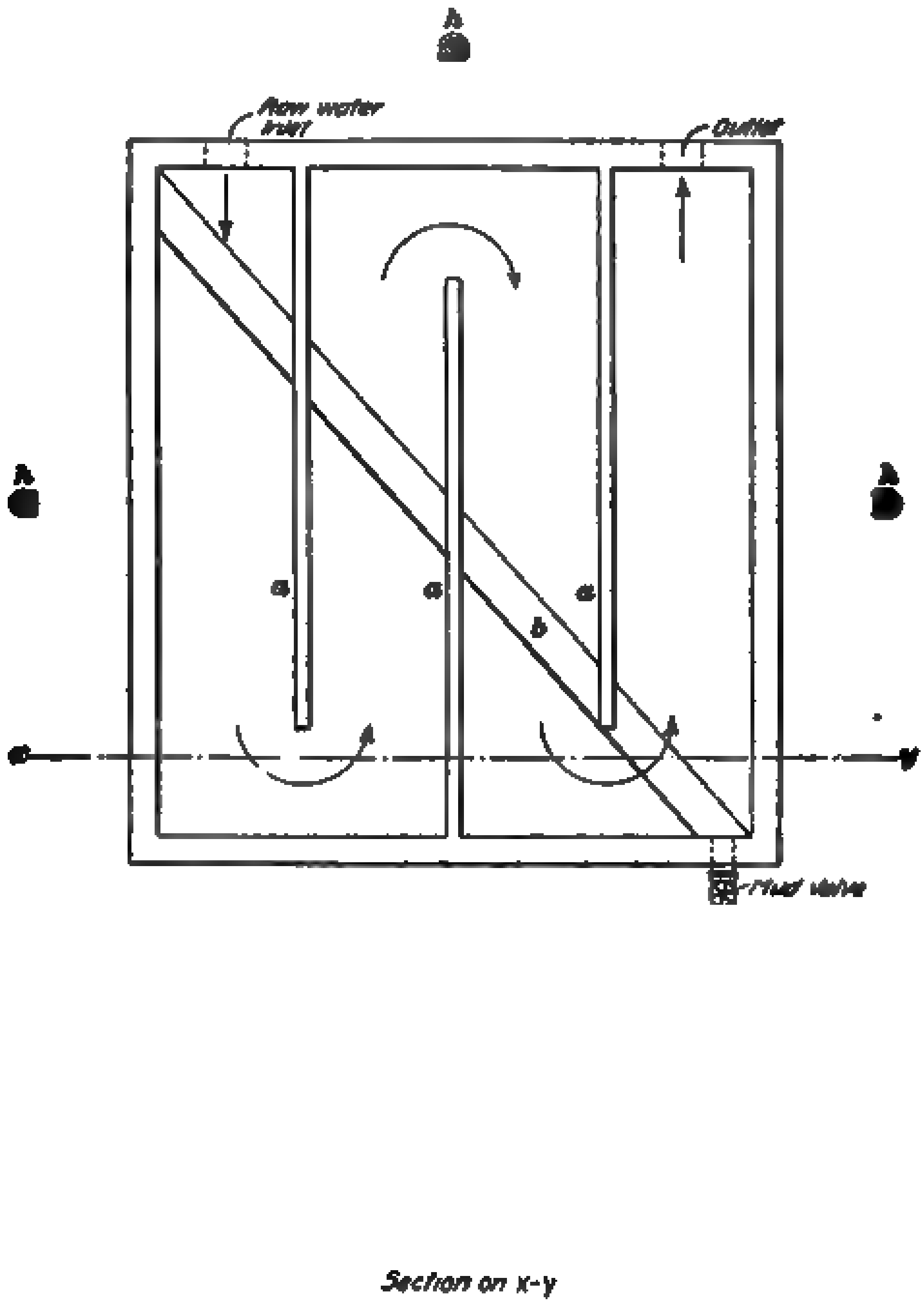


FIG. 2

baffle walls and has the outlet pipe at the opposite corner from the inlet pipe.

53. **Shape and Material.**—The shape of and materials for settling basins are determined mainly by local conditions.

Usually, the shape depends on the configuration of the ground, but, when there are no limitations, the basin is made circular or square.

Generally, settling basins are made of concrete or of stone masonry; sometimes, they are made with earthen embankments puddled with clay and having sloping sides paved with cobblestones. No fixed rules can be laid down for materials, the selection of which depends to a great extent on cost. When puddled-clay settling basins are to be constructed, the site should be cleaned of vegetation, and the surface soil removed for a sufficient depth to clear the site of all loam and organic substances.

54. Velocity in Continuous-Flow Settling Basins. In continuous-flow basins, the turbidity of the water determines the length of the basin. The necessary velocity of flow depends on the turbidity of the water to be treated, and on whether the settled water is to be subsequently filtered. If the water is to be filtered, the same degree of clarification will not be required as where sedimentation is the only treatment. It may be stated that, as a rule, clear lake or reservoir water requires no treatment preliminary to filtration. It is only the waters from running streams, which permanently or at times are turbid, that require sedimentation. As explained in Art. **5**, the turbidity of streams differs, and it is obvious that a longer flow would be required to treat the waters of a muddy stream that carries in suspension large quantities of finely divided clay or other matter that clears slowly, than would be required for the treatment of water that carries less matter in suspension and clears more rapidly.

55. The velocity of flow through a continuous-flow basin must be sufficiently slow not to cause eddies, which would interfere with proper sedimentation. The permissible velocity is variously estimated at from 2 to 5 inches per minute. When designing a settling basin, it is well to take into consideration the future growth of the city or town, and construct the basin of a length based on a velocity of about 2 inches per minute. By increasing the velocity within safe limits,

the original basin can be made to do service in future years, when there may be a demand for a greater supply of water.

If provision is made for a sedimentation of 24 hours, with a velocity of 2 inches per minute, the channel of a settling basin will have a length of 240 linear feet, and a cross-section capable of supplying the required amount of water at the stated velocity.

56. Depth of Settling Basins.—The principal conditions that limit the depth of a settling basin are the bearing capacity of the soil, provision for deposits, and removal of mud. The depth should be such that the bottom velocity will not carry out of the basin too much sediment. When space will permit the construction of shallow basins, that is, basins of depths from 10 to 16 feet, they will be found the most satisfactory in operation, as their small depth facilitates the removal of sediment. When, however, space is limited, equally good results in the clarification of water can be secured by constructing deep settling basins. A depth of from 3 to 4 feet is allowed in the bottom of all settling basins for the accumulated deposits of sediment. This allowance is included in the depth of the basin, which for efficient sedimentation should not be less than 10 feet.

57. Location of Settling Basins.—The location of a settling basin depends greatly on local conditions. When a city is to be supplied with water through a gravity system, and there is a hill convenient for the location of a settling basin, such a situation will be found the most satisfactory. The raw water can be pumped directly from the river to the settling basin, and means may be provided for disposing of the sediment by gravity.

58. Cleaning Settling Basins.—The frequency with which settling basins should be cleaned depends on the amount of water used, the proportion of suspended matter, and the depth of mud provided for in the settling basin. Usually, a depth of 4 feet is allowed for the deposit of mud; when that limit is reached, the water is drawn off, and the settling basin cleaned. The mud is removed in barrows or

buckets, and the sides and bottom of the basin are washed with water from hose attached to hydrants placed around the sides of the basins for that purpose. The mud and water from the washings are carried out through a mud-valve toward which the bottom of the basin slopes.

Settling basins that are located at a lower level than the river are provided with a sump into which the sediment is washed through the mud-valve. The mud is subsequently removed by a centrifugal pump or other mechanical appliance.

59. Clear-Water Well or Reservoir.—When water from a continuous-flow settling basin is supplied by gravity directly to the consumer, it is well to interpose a clear-water basin at a lower level than the settling basin, so that the clarified water can discharge into it by gravity. A clear-water basin is required only to store sufficient water to compensate for the hourly fluctuations due to irregular consumption, and for this purpose need have a capacity of only 25 per cent. of the daily consumption.

When a settling basin is located at a low elevation and water passes from it to the pump, it is necessary to provide a suction well into which the clear water can be discharged and from which it can be drawn by the pumps. For most installations, a suction well with a capacity of 1 hour's supply will be found sufficiently large.

COAGULATION

60. As applied to the purification of water, **coagulation** is effected by adding to the impure water certain substances, called **coagulants**, that act chemically either on each other or on the impurities present in the water. The result of such reactions is a flocculent substance that gathers to itself the finely divided particles of sediment, and thus draws them into groups or aggregates that are more easily removed by subsequent treatment than are the individual particles. Bacteria, if present, are drawn together by the coagulation and are removed with the other suspended matters.

61. The substance most commonly employed as a coagulant is aluminum sulphate, but compounds of iron are also used. A solution obtained by dissolving scrap iron in water that has been charged with the fumes of burning sulphur has been sometimes used. This solution not only purifies the water mechanically, but also removes certain colors. When aluminum sulphate is added to impure water, it is decomposed, and a light flocculent precipitate of aluminum hydrate is formed, which, gradually settling to the bottom of the reservoir, gathers the suspended matters and also certain dissolved impurities and carries them down mechanically. A disadvantage of this method of purification is that, through the decomposition of the aluminum sulphate, sulphuric acid is formed, which is an impurity that must be removed. If the water contains sufficient lime, however, the sulphuric acid combines with the calcium and forms calcium sulphate, which is dissolved by the water. If lime is not present in the water in sufficient amount to combine with the sulphuric acid, more lime may be added. The disadvantage of this is that the formation of calcium sulphate increases the hardness of the water.

62. Flood waters usually require the most coagulant for their treatment, and also contain the smallest amount of lime or alkalies for combining with the sulphuric acid. Many waters contain sufficient lime to combine with the acid of the necessary coagulant at all times. Other waters are deficient in lime at times of flood only, and still others, particularly very soft waters, if they are highly colored, are normally deficient in alkalinity. For such waters, it is necessary to add lime constantly in connection with the coagulant. Soda ash can be used in place of lime to increase the alkalinity; it is more expensive than lime, but it does not harden the water.

63. The amount of coagulant required to remove a given amount of turbidity or a given amount of color depends on the amount and character of the matters to be removed, and is very different in different cases. The

proper amount of coagulant to be used in a given case can often be approximately ascertained from certain available data determined by an expert, but the methods of computation are beyond the scope of this work. If too small an amount is applied, it is usually entirely inefficient, while little, if any, advantage is obtained from the use of an excessive quantity. The amount required often fluctuates from day to day, and even from hour to hour, with the quality of the raw water, and much skill is required to properly regulate it.

Waters containing large quantities of suspended matters may absorb and render ineffective part of the coagulant, and it will then be necessary to use an additional amount. With such waters, it is usually advantageous to remove as much of the suspended matter as possible by sedimentation before applying the coagulant. After the coagulant is applied, a certain length of time must be allowed for the chemical changes to take place and for the precipitate to collect. The length of time required decreases as the alkalinity of the water increases, and lime is often added to shorten it. Economical periods of coagulation may range from 1 to 12 hours.

PURIFICATION OF WATER

(PART 2)

WATER FILTRATION

INTRODUCTION

1. Filtration consists primarily in passing water through a porous substance that intercepts and retains the suspended matters contained in the water. In filtration as actually practiced, however, many other actions take place, which are sometimes even more important than the primary action of straining. For the purposes of filtration, special reservoirs or receptacles, called **filters**, are constructed.

2. Kinds of Filters.—There are two principal types of filters; namely, *slow sand filters*, and *rapid*, or *mechanical*, *filters*.

3. A **slow sand filter** consists of a reservoir or tank having a bed of sand through which the water to be treated slowly percolates into a system of underdrains below. This system of underdrains collects the filtered water and conducts it either to the distributing main or to a suitable storage reservoir. The efficiency of a slow sand filter is due not so much to the mechanical straining of the water through the sand, as to a biological process that takes place in a jelly-like film that is formed on the surface of the sand. This film, which is teeming with bacteria, is pervious to the water, but entangles and holds back any solid substance that comes in contact with it, and, if the matter so held back is of organic

origin, it is quickly destroyed by the micro-organisms in the jelly layer. When this jelly, or sediment, layer becomes so thick and dirty, from the impurities it contains, that sufficient water cannot pass through the filter, the water is drawn off, and the bed cleaned by scraping off a thin layer of sand.

4. A **mechanical filter** consists of a bed of sand, in a vat or tank, a system of underdrains below the sand to collect the filtered water, and a mechanical rake to agitate the sand bed while the filter is being washed. Mechanical filters are washed by forcing filtered water, and sometimes filtered water alternating with compressed air, through the bed from the underdrains.

In mechanical filters, a sediment layer is artificially produced by the use of coagulants. Clarification of water can be effected by passing it without coagulants through a mechanical filter; but, when perfectly pure water is desired, coagulants are used.

5. Sand Bed.—As already stated, sand is the material forming the filter bed through which the water is strained. Many other materials have been proposed, but nothing has been found to take the place of sand. The sand grains must be of uniform size, and neither too fine nor too coarse. If they are too fine, sufficient water cannot pass through without too great loss of head. If, on the other hand, the sand is too coarse, the purification obtained will be inadequate. Practical experience shows that the range in sand sizes that can be used is comparatively limited. The size employed corresponds approximately with that of good mortar sand, but the size of grains should be determined by trial and accurate measurement before use. Carefully measured screens are used in determining the sizes of the sand grains, and in some cases other and more complicated methods of measurement are resorted to. The thickness of the sand layer in a filter is usually between 2 and 4 feet.

6. Rate of Filtration.—The rate at which water is passed through the sand varies greatly. The straining is much more thorough at low rates. If even a very bad water

is passed through clean sand at a very low rate, it will be well purified. Practically, rates of filtration less than 2,000,000 to 3,000,000 gallons per acre of filter surface daily, or from 3 to 5 vertical inches per hour, are seldom used. If a water contains matter that cannot be removed by filtration at these rates, preliminary processes are more economical than the use of a lower rate. At the rates mentioned, with properly constructed filters, very nearly all the bacteria and practically all the turbidity can be removed from waters that are not exceedingly turbid. Waters containing on an average more than 50 parts per million of suspended matter cannot be successfully treated in this way continuously, because the suspended matters will clog the filter too rapidly, necessitating too frequent cleanings. When a water has been coagulated, it can be filtered at a much higher rate without much sacrifice in efficiency. This is because the minute particles have been drawn into groups of such size that they are more readily removed. In designing filters, the rate of filtration is commonly assumed as 3,000,000 gallons per acre daily. If an attempt is made to filter an unusually bad water without coagulation, a lower rate should be assumed; and for some lake and reservoir waters it is safe to assume a higher rate.

7. Biological Nature of Filtration.—The changes effected in water by sand filtration are much more radical than can be accounted for by simple straining. It has been found that many organisms establish themselves on and become attached to the grains of sand in the filter; they live on organic matter contained in the passing water. These organisms are not present in a filter just constructed, and some time is required for them to become established. For this reason, a new filter gives less perfect purification than one in regular service; and, if the operation of a filter is suspended for some time, the organisms die or become inactive, and a little time must be allowed, after the filter is again put in service, for them to regain their normal activity.

8. Capacity of Filters.—A sufficient area must be provided so that some filters can be put out of service for the

purpose of cleaning, and a reserve filter capacity must be provided sufficient to meet the maximum draft, and not merely the average annual use. The excess of filter capacity over the average annual use depends entirely on the amount of reservoir capacity between the filters and the consumers. If the filtered water flows or is pumped to ample distributing reservoirs, filters may be provided to meet only the maximum weekly consumption. If the direct system of pumping is used, and if there are no reservoirs, the filter capacity must be relatively greater; but in this case it will pay to put a pure-water reservoir near the filters, with a capacity of from one-fourth to one-half of the maximum daily consumption, to balance the hourly fluctuations in the rate of consumption and allow a steady rate of filtration to be maintained. This reservoir should be covered, to avoid the growth of microscopic organisms.

CONSTRUCTION OF SLOW SAND FILTERS

GENERAL CONSTRUCTIVE FEATURES

9. When slow sand filters are constructed in wet land, the site should be provided with a system of subsoil drains to prevent the ground water from rising to and mixing with the pure water in the filter underdrains. Also, the floor and walls of the filter should be well **puddled** with clay; that is, lined with clay in the form of a thick paste, or otherwise made water-tight, so that the filtered water cannot escape to the subsoil drains. If the ground water from the subsoil drains cannot be discharged by gravity, it may be gathered in a sump and discharged by mechanical means into the nearest watercourse or sewer.

10. Filters are made water-tight in various ways. The simplest method employed is to puddle the bottom and sides of the filter with clay. In some cases, the bottom and sides of a filter are paved with stones or bricks on top of the clay, and in most cases the sloping sides are paved to protect them

from the abrasion of ice. In all puddled or paved filters, the sides are sloped from 2 to 3 feet horizontal to 1 foot vertical.

The best types of open filters have the bottom and sides constructed of concrete or other masonry. The walls are carried up vertically, with a ledge, 4 inches wide and a little above the gravel, for the sand to rest on and form a closer joint than it would with the vertical walls.

The shape of a filter depends on local conditions; the usual shape is rectangular, although no good reason can be given why a filter should not be made any other shape. The depth averages about 10 feet, proportioned as follows: 6 inches for underdrains, 1 foot for depth of gravel, 4 feet for maximum depth of sand, and 4 feet 6 inches for the depth of water.

11. Pure-Water Reservoirs.—At all filtration plants, pure-water reservoirs should be provided to store a supply of water to compensate for the hourly fluctuations due to an irregular consumption.

Filtered water deteriorates with storage; it is desirable, therefore, to deliver it to the consumer as soon as possible. To insure a quick delivery to the distributing mains, pure-water reservoirs should not exceed in capacity 25 per cent. of the daily water consumption.

Filtered water should be protected from contamination while in storage by covering the reservoir to exclude dust, light, and insects; and the roof of the reservoir should be plentifully supplied with ventilators.

UNDERDRAINS

12. The underdrains for slow sand filters are a series of agricultural, or field, tile pipes placed on the bottom of the filter to collect the water that percolates through the filter bed. These drains are laid with open joints, and the pipes may be perforated to provide additional openings for the inflow of filtered water. The trunk line of the main drains connects to the discharge pipe from the filter.

There are two principal systems of underdrains used with slow sand filters. The first system, which is shown in Fig. 1, consists of a main trunk line or pure-water collector *a*, to which is connected a system of branch drains *b*, spaced about 30 feet apart. Each drain is extended to within about 4 feet of the filter wall, the end being closed with a stone. If the pipes are not perforated, the joints between them must be

FIG. 1

left open for the entrance of the filtered water. The spaces between the drains are filled with broken stones. In a system of this description, the pipes should be large enough to carry off easily the greatest amount of water that is filtered. The branch lines for small filters may be of 4-inch pipe, and the trunk lines of 6- or 8-inch pipe. For large filters, the branch underdrains may be of 6- or 8-inch pipe, and the collector so proportioned to the amount of water it must carry that there will be little loss of head due to the underdrains.

13. The second system of underdrains, shown in Fig. 2, consists of a flooring of hollow perforated tile covering the entire filter bottom. These drains are laid with slightly open joints, and the gravel layer is placed on top of the tile, the perforations being small enough to prevent the gravel from falling through into the underdrains. For this system, the tile collectors are generally extended to the walls of the filter. A better method, however, is to end the drains a few feet from the walls and fill the space with sand, so that water passing down the inner surface of the walls of the filter will have a slight lateral movement through the sand from the filter walls to the underdrains. This precaution will insure a purification of any raw water that, owing to cracks or imperfect construction, may pass through the filter walls.

FIG. 2

FILTER BEDS

14. The bed of a slow sand filter suitable for general purification purposes is shown in Fig. 1. It consists of a layer of fine sand, about 4 feet thick, supported by a bed, about 1 foot thick, of graduated sizes of gravel; this bed of gravel rests on the underdrains. Crushed stones are placed on the bottom between the underdrains and covering them to a depth of a few inches, and assorted sizes of gravel are placed in about 2-inch layers on top of the crushed stone. The largest size forms the first layer, and the last layer on top is of very fine gravel capable of supporting the sand bed without sand grains falling down and clogging the gravel layers.

A bed of sand 4 feet thick is not necessary for perfect filtration, but the thickness of the bed gives stability to the filter and reduces the possibility of raw water breaking through to the underdrains. It also allows repeated scrapings of the bed for cleaning purposes. When the sand has

been reduced by repeated scrapings to 12 inches in thickness, the water should be drawn off from the filters, an extra deep layer of sand removed, and the sand that has previously been removed should be washed and returned to the bed. The filter should then be filled by allowing water to rise from the underdrains until it covers the surface of the sand about 6 inches, after which raw water can be admitted through the inlet.

The coarse gravel at the bottom of a filter bed has but little effect in the process of purifying the water; it acts chiefly as a porous stratum through which the pure water passes to the underdrains. The graduated sizes of gravel or sand that are laid between the coarse gravel and the fine-sand bed on top are used only to prevent the fine filter sand from washing down and clogging the gravel bed.

15. Quality of Sand.—Filter sand should be free from clay, loam, vegetable matter, or lime. Clay or loam will cement the grains of sand together and cause subsurface clogging of the filter. Vegetable matter will contaminate the bed, and lime sand in sufficient quantities will harden the water. Vegetable matter, loam, and clay can be removed by washing, and, if sand perfectly free from all impurities cannot be procured from a sand bank or river bed, it should be washed before being used. If the amount of lime in the sand is not excessive, the sand may be used, as the degree of hardness will decrease with age, but if pure soft water is wanted, sand containing lime should be rejected.

16. A simple test for lime in sand is to wet the sand with hydrochloric acid; if it gives off a gas, this indicates the presence of lime, the amount of which can be judged by the quantity of gas given off and the appearance of the samples after the test.

17. Effective Size of Sand.—The size of sand used for filter beds varies from the finest obtainable to the coarsest kind. The sand must all be sifted, and deposited in consecutive layers, the coarsest at the bottom and the finest at the top, because the filtration is downwards. The practical

objection to the use of very fine sand is that it clogs much quicker than coarse sand, and requires more frequent scrapings. The coarse sand, on the other hand, allows the sediment to penetrate deeper, and the extra sand that must be removed in scraping will offset the more frequent cleanings of the fine sand.

18. In comparing various sands for filtration purposes, the size of a sand grain is classified or rated by the diameter of a sphere of equal volume, regardless of the shape of the sand grain. The sand grain can be measured on three dimensions, one being its greatest dimension and the other two being at right angles to the greatest dimension and to each other, and the cube root of the product taken as the diameter of the corresponding sphere. In practice, for average waters, a sand $\frac{1}{75}$ inch in diameter has been found to give the best general results. For comparatively clear lake or reservoir water, a fine sand about $\frac{1}{150}$ inch in diameter can be used, while a coarse sand about $\frac{1}{64}$ inch in diameter will probably be found best for turbid river waters. It has been found that, when the particles of the filtering material are of different sizes, it is the finer particles, occupying the intervening spaces between the larger particles, that chiefly determine the character and effectiveness of the material for filtration purposes. In order to have a basis of comparison for the sizes of different sands, a size such that 10 per cent. by weight of the grains composing a given mass of sand are smaller than it, and 90 per cent. of the grains in the mass are larger than it, is referred to as the **effective size** of the sand in the mass.

19. Uniformity of Sand Particles.—An important condition with regard to sand used for filtering is its degree of uniformity. The sizes of the sand particles may be nearly uniform or may vary greatly, and this will affect the effectiveness of the filter very materially. The variation in the sizes of the sand grains in a given sample is expressed by what is called a **uniformity coefficient.** This is the ratio of the size of a large sand grain to that of a small sand grain,

each grain being of a certain definite size with respect to the other grains of the sample. The large grain is of such a size that 60 per cent. by weight of the grains of the sample are smaller than it, and the small grain is of such size that only 10 per cent. of the grains of the sample are smaller than it.

20. Resistance of Sand to Flow of Water.—The frictional resistance of closely packed sand to the passage of water through it in such quantity as to completely fill the spaces, and when the sand is free from clogging, can be found from the formula

$$v = k d^2 \times \frac{h}{l} \times \frac{t + 10}{60}$$

in which v = velocity or rate of filtration, in meters per day, or, approximately, in million gallons per acre* per day;

k = a constant, to which experiments give a value of approximately 1,000;

d = effective size of sand grain, in millimeters;

h = loss of head due to resistance of sand;

l = depth of layer of sand through which water passes, stated in same unit as loss of head;

t = temperature, in degrees Fahrenheit.

The loss of head represents the resistance of the sand. As used in the formula, it is always stated in the same unit as the depth of the sand. The quantity $\frac{h}{l}$ may, therefore, be considered as a ratio, and is often so taken. When the loss of head is the unknown value to be determined by the formula, the result is always in the same unit as the depth of the sand. This formula should be used only for sands having uniformity coefficients below 5 and effective sizes from .1 millimeter to 3 millimeters.

EXAMPLE.—The depth of the layer of sand in a filter is 39 inches, and the effective size of the sand grains is .32 millimeter. When the temperature is 50° F., the rate of filtration is 1.6 million gallons per

*One meter in depth over an area of 1 acre is equivalent to 1,069,012 gallons.

acre per day. What is the loss of head due to the frictional resistance of the sand: (*a*) in inches? (*b*) in millimeters?

SOLUTION.—(*a*) By substituting the given values in the formula,

$$1.6 = 1{,}000 \times .32^2 \times \frac{h}{39} \times \frac{50 + 10}{60};$$

whence $h = .609$ in. Ans.

(*b*) Since there are 25.4 millimeters in 1 in., very closely, the value of h, in millimeters, is

$$.609 \times 25.4 = 15.47. \text{ Ans.}$$

21. Resistance of Gravel to Lateral Flow of Water. After filtering downwards through the sand, the water must flow in a nearly horizontal direction through the layer of gravel in order to reach the underdrains, and to this horizontal flow the gravel offers some resistance. For the purpose of calculating the loss of head due to the resistance of the gravel to the flow of the water through it at the very

TABLE I

VALUES OF DISCHARGE COEFFICIENT

Effective Size of Gravel		Discharge Coefficient
Millimeters	Inches (Approximate)	c
5	$.1969 = \frac{1}{5}$	23,000
10	$.3937 = \frac{2}{5}$	65,000
15	$.5906 = \frac{3}{5}$	110,000
20	$.7874 = \frac{4}{5}$	160,000
25	$.9843 = 1$	230,000
30	$1.1811 = 1\frac{1}{5}$	300,000
35	$1.3780 = 1\frac{2}{5}$	390,000
40	$1.5748 = 1\frac{3}{5}$	480,000

low velocities common to filters, the following approximate formula may be used:

$$h = \frac{v b^2}{2 l c}$$

in which h = loss of head, in feet;

v = rate of filtration, in million gallons per acre per day;

b = greatest horizontal distance, in feet, through which water flows;

l = average depth of gravel, in feet;

c = discharge coefficient.

Where the underdrains are parallel, as is usually the case, b is taken as one-half the distance between the drains.

The discharge coefficient c for any gravel is equal to 1,000 times the quantity of water, expressed in million gallons per acre per day, that will pass when $\frac{h}{l} = \frac{1}{1,000}$. In Table I are given approximate values of this coefficient for gravels of different sizes.

EXAMPLE.—The rate of filtration is 1.15 million gallons per acre per day, and the water flows to parallel underdrains 20 feet apart through a layer of gravel uniformly 1 foot in depth and having an effective size of 25 millimeters. What is the loss of head in the gravel?

SOLUTION.—From Table I, the discharge coefficient for gravel having an effective size of 25 millimeters is found to be 230,000. By substituting this and the given values in the formula, the loss of head is found to be

$$\frac{1.15 \times 10^2}{2 \times 1 \times 230,000} = .00025 \text{ ft. Ans.}$$

22. Loss of Head.—Loss of head in a filter is the amount of head required to force the water through the filter bed. With a filter just cleaned, this amounts to 1 or 2 inches. As the work of filtering proceeds, the surface layer of sand becomes dirty, and more head is required to force the water through. The loss of head gradually increases to the maximum limit allowed, when the filter must be thrown out of use and cleaned. Four feet loss of head is commonly allowed. In some European works, the limit is 2½ or 3 feet. In many of the older works, it was 6 feet; and it will sometimes facilitate operations to have the plant constructed so that a large amount can be used if necessary. Allowance is usually made for a depth of from 3 to 4 feet of water above the sand in the filter. With open filters, the water must be deep enough so that the sand will not be disturbed by cutting and removing the ice. It is also customary to make the depth of water equal to the maximum loss of head that it is

intended to use in operating the filter, so that all the head on the sand will be what is called the positive head, that is, so that the pressure of the water on the sand will never be less than the resistance that the sand offers to the passage of the water through it.

23. Laying Sand Courses.—To insure a uniform rate of filtration throughout the filter bed, it is necessary so to deposit the sand that it will have a uniform density and depth. This cannot be done by laying it in thin layers, or in one layer spread over the entire surface of the filter bed. The best way to deposit the sand is in two or three layers, the full width of the filter, each layer being laid continuously across the filter the full thickness of its bed. Planks should be placed for the workman to walk on, and the surface of the sand should be well raked when the planks are removed.

In placing the gravel around the underdrains, care should be taken to bury the lateral drains under at least 6 inches of gravel, and fill the space between the drains to the same depth. The gravel should stop about 3 or 4 feet from the side walls, and the remaining space should be filled with sand. The gravel should be well packed down and settled before the sand is placed, to prevent disturbance of the sediment layer on top of the fine sand by a subsequent settlement of the gravel.

The part of a filter most likely to afford a passage for unfiltered water is in or around the vertical walls. To guard against this many expedients are resorted to. Stepping the walls and stopping the underdrains 3 or 4 feet from the filter walls have already been mentioned. Other precautions that are found effective are sanding the walls or washing them with a coat of Portland cement. It is not good practice to plaster stone or brick walls with cement below the water-line, as cement sometimes adheres in spots only, and water entering to the back of the plaster through a crack can then work its way unfiltered to the bottom of the filter bed.

Brick or stone walls or piers are not so suitable as concrete for the construction of filters below the water level. There

is seldom a good joint between the mortar used and the brick or stone, and unfiltered water entering a joint can follow down to the bottom of the filter, and thence to the underdrains.

RAW-WATER INLETS

24. Hand-Operated Inlet.—Inlets for the admission of raw water to slow sand filters are made to operate either automatically or by hand. A simple form of hand-operated inlet is shown in Fig. 3. It consists of a 90° bend *a* that

FIG. 3

turns into a masonry chamber *b* on the inside of the filter *c*. The rate of flow to the filter is controlled by partly closing or opening the valve *d*. By paving the filter bed around the inlet chamber with brick, as shown at *e*, the surface of the sand is protected from being washed away. This form of inlet requires constant attention to prevent a fluctuating water level in the filter, which would cause a corresponding variation in the rate of filtration.

25. Automatic Inlets.—To prevent fluctuations in the filter water level, automatically regulated inlets may be used. Fig. 4 shows a simple form of **automatic inlet regulator.**

It consists of a vertical cylinder *a* attached to the inlet pipe *b*. A vertical rod *c*, to which two disks *d*, *e*, and a float *f* are attached, passes through the cylinder. The rod *c* is held in position by the guides *g*, *g*. When the water in the filter reaches the high-water level, the float raises the two disks against the seats *h*, *i*, thus shutting off the water. When the

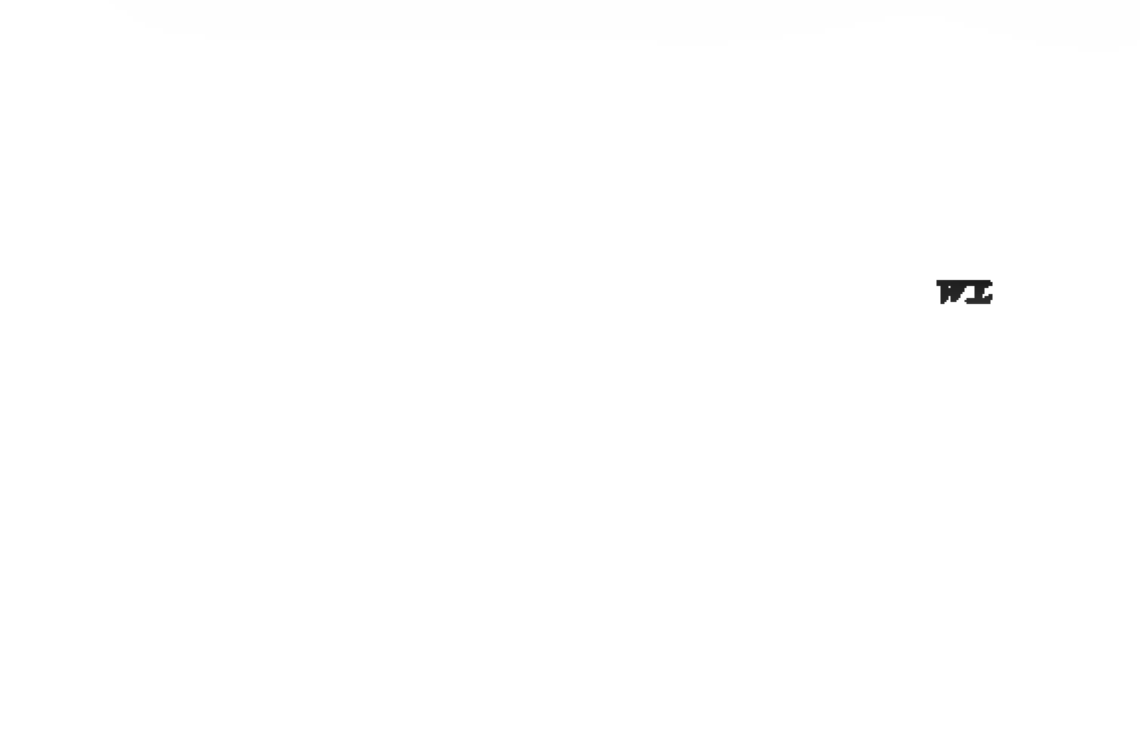


FIG. 4

water in the filter is lowered, the float descends, thus opening the valve for the admission of raw water. In this manner, an almost constant water level is maintained, the fluctuations rarely exceeding 6 inches.

REGULATION OF OUTFLOW

26. Filters should be provided with a regulating apparatus through which the water should flow on leaving the filter. This apparatus should indicate the rate of filtration and the loss of head, and should also be able to control both. Many forms of apparatus are used for this purpose; some involve the use of weirs, submerged orifices, and Venturi meters. A regulating apparatus should be so designed as to allow the filter to be entirely drained for the purpose of inspection.

to allow the filtered water to be wasted, and to allow each filter, after scraping, to be filled from below through the outlet pipes with filtered water from some other filter, in order to avoid the disturbance caused by taking raw water over the sand immediately after a filter has been drained.

The rate of filtration in slow sand filters may be regulated in any one of three ways. In the first method, the rate of filtration depends on the rate of consumption. The pure-water reservoir is built on a level with the filter, and the water in the two compartments is practically on the same level. When the consumption of water is light, the rate of filtration is low. When the consumption of water is excessive, as may be the case during certain hours of the day, the rate of filtration is correspondingly increased, and the water that passes through the filter during such periods is liable to be below the average in clearness and purity.

27. The second method of control, the principle of which is shown in Fig. 5, involves operation by hand, and is

FIG. 5

entirely independent of the rate of consumption. Pure water enters the **effluent chamber** *a* from the filter *b* through the underdrain *c*. When the sand bed is clean and the water

passes through with but slight resistance, a gate *d* is raised to decrease the head *e* that forces the water through the sand. As the surface of the filter bed becomes clogged with sediment, a greater head is necessary to force the required amount of water through, and the gate is consequently lowered. When the gate has been lowered to such a level that the head, or the difference between the level of the water in the filter and the level of the water in the effluent chamber, is 6 feet, the filter must be emptied and the filter bed cleaned. Any further loss of head is liable to cause a break in the sand and allow unfiltered water to pass through to the underdrains. The filtered water that passes over the gate from the effluent chamber is collected in the pure-water reservoir *f*, which is located at a lower level than the filter. A crank operating a rack and pinion serves to raise and lower the gate, and a ratchet wheel *g* and dog *h* locks it in place.

28. In the third method, the rate of filtration is regulated automatically and independently of the rate of consumption, as shown in Fig. 6. A float *a* is attached to a telescopic

FIG. 6

cylinder, or joint, *b*, which moves up and down over a pipe *c* as the water in the effluent chamber *d* fluctuates. The telescopic joint is slotted on the sides near the top to serve as an outlet for the water. When the filter bed is new and the rate of filtration rapid, the telescopic joint and float are

automatically raised by the water that rises in the effluent chamber. As the surface of the filter bed becomes clogged, the float and outlet are automatically lowered by the fall of water in the effluent chamber; but, as the outlet openings in the side of *b* are maintained at a fixed distance below the float, a uniform rate of filtration is maintained. The rate can be increased by weighting the float so as to lower the outlet openings, and conversely the rate can be decreased by attaching a counterweight *e* to the chain to raise the outlet. In the latter case, the counterweight may be used as a gauge to register the loss of head by attaching a scale *f* and an indicator arm, as shown.

COVERING OF FILTERS

29. When open filters are used in cold climates, ice forms over the entire surface of the water. If there are but two filters that operate alternately, the bed of one must be scraped while the other is supplying water. Scraping filter beds in freezing weather is exceedingly inconvenient, owing to the necessity of removing the ice; besides, it has been observed, although the cause of the phenomenon is not well understood, that after a filter has been scraped in freezing weather, its efficiency is considerably impaired. To avoid the expense of removing the ice, as well as the diminution of efficiency following scraping, enough filter beds are sometimes supplied to tide over periods of excessively cold weather without scraping the filter beds.

30. Most of the slow sand filters are built without protection from the weather. While this practice is generally safe in climates where ice does not form on the surface of the water during winter, there are certain conditions under which covered filters are advisable even in such climates. When the supply is derived from ground water or from other sources that contain much mineral matter, the water should be filtered and stored in the dark to prevent the development of an objectionable water vegetation, known as **algæ**, that rapidly clogs the surface of the filters

Fig. 7

and imparts a disagreeable taste and odor to the water. So rapidly does vegetation develop in some ground waters that the periods between scrapings of filter beds are thereby reduced by one-half, and so thick and matted are some of the growths that they can be rolled up like a carpet.

31. In climates where the temperature remains below the freezing point for long periods of time, and a thick layer of ice forms on all exposed waters, it is advisable to cover filters, both for sanitary and economical reasons. Filters may be covered with arches of masonry or with wooden roofs, according to the requirements in each particular case. In warm climates, where a roof is used only to darken the reservoir and prevent the growth of algæ, a wooden construction will be suitable; but in cold climates, where the object of filter covers is chiefly to prevent the freezing of the water, the cover should be made of masonry, and the top covered with several feet of earth. Arches, either domed or elliptical in shape, and built of brick, hollow tile, or cement concrete, are generally used. The cost of constructing covered filters of masonry averages about 50 per cent. more than the cost of constructing open filters under similar conditions. The saving, however, in the cost of operating covered filters in cold climates will more than compensate for the extra first cost, and such filters are preferable for all localities where the mean winter temperature is below 30° F.

32. In Fig. 7 is shown the interior of part of a covered filter. The drains are all in place and are covered with a layer of coarse stones. The layers of finer gravel and sand are not in place. The filter bed when in place will finish even with the top of the circular brickwork in the corner, which contains the raw-water inlet. The circular tank shown within the inlet chamber is a large float that closes a balance valve and thus shuts off the inflow of raw water when the water level in the filter reaches the desired height. Air for ventilation is admitted through openings in the masonry cover.

OPERATION OF SLOW SAND FILTERS

33. Filling a Filter.—When filling a filter after the bed has been scraped, the water should at first be admitted through the underdrains until the sand is covered to a depth of at least 6 inches, and then water may be admitted through the raw-water inlet. This method prevents the disturbing of the sand that occurs when an empty filter is filled entirely through the raw-water inlet. When filling the filter through the underdrains, filtered water is used, the supply being taken from one of the other filters through a by-pass that connects the underdrains. When the by-pass valve is opened, water from one filter will rise through the bed of the other filter to the required level.

34. Cleaning a Filter Bed.—The efficiency of a filter depends largely on the fineness of the filtering medium. When the sand in the filter bed is supplemented by a fine layer of sediment on the surface of the sand, the efficiency of the filter is increased and continues to increase with the growth in thickness of the sediment layer until this layer becomes too thick for sufficient water to pass through. When the surface clogging reaches such a stage, the water must be drawn off from the filter, and the sediment and top layer of sand removed by scraping.

The length of time required for a sediment layer to become so thick as to require cleaning depends on the kind of water to be purified and on the rate of filtration. Thus, with very turbid water, the filter beds will probably require cleaning once a week, while with comparatively clear water the periods between scrapings may exceed 60 days. It has been found in practice that for average waters the length of time between scrapings is about 20 days.

35. Filters are usually scraped with broad shovels, the dirty sand being removed in wheelbarrows running on planks. The average depth of the sand removed is $\frac{3}{4}$ inch, or approximately 100 cubic yards of sand per acre of filter surface. The depth of the sand in the filter should be such

that it can be scraped ten to twenty times before replacing the sand, without reducing too much the thickness of the layer. The last scraping before replacing the sand should be deeper than the ordinary scraping. A sand ejector is now used for removing sand from covered filters. As the sand or sediment layer is scraped, it is shoveled into a hopper from which it is carried by a jet of water, through a hose, to the sand washers.

(a)

36. Washing Filter Sand. The sand removed from a filter bed by the process of scraping is not thrown away, but is washed and stored ready to be used again. Formerly, filter sand was washed by turning a stream of water on it from a hose. That method, however, is so unsatisfactory that it has been entirely abandoned, and the sand is now washed in some type of sand-washing machine. An apparatus designed for this purpose is shown in Fig. 8 (*a*) and (*b*). It consists of four hoppers *a*, *b*, *c*, *d* supported by a frame work and supplied with water through 1½-inch inlet pipes *e*, *f*, *g*, *h*. Each hopper outlet is provided with an ejector nozzle similar to the one shown in Fig. 8 (*b*). From the outlet

(b)

FIG. 8

nozzle of each hopper, a waste pipe extends over and discharges into the next adjoining hopper. Overflows *j*, *k*, *l*, *m* discharge into a trough that empties into a sump connected to the sewer. The operation of the washer is as follows: Dirty sand is shoveled into hopper *a*, falls through a wire screen to the bottom and is carried through the outlet nozzle and pipe *n* into the hopper *b*. The action of the screen in the hopper *a* and of the jet of water separates the grains of sand and subjects each grain to the scouring action of water. As the dirt is lighter than water, it floats to the top of the hopper *b*, and is carried through the overflow *k* to the sewer. The sand that settles to the bottom of the hopper *b* is ejected by the nozzle at the bottom into the hopper *c*, where the same operation is repeated that took place in hopper *b*. From *c*, the sand is ejected to *d*, and from there to the sand trough, each succeeding operation still further cleaning the sand and separating the dirt. From the sand trough, the washed sand is conveyed to storage bins for future use.

MECHANICAL FILTERS

CLASSIFICATION

37. There are two kinds of mechanical filters: *gravity filters* and *pressure filters*. **Gravity filters** are open to the atmosphere at the top, and the water percolates through the filter bed by gravity. **Pressure filters** are closed watertight vessels into which the water is forced through the filter bed by the full hydraulic pressure in the service pipe. The method of filtration is the same in gravity as in pressure filters. Raw water enters the filter at the top, passes down through a bed of sand, and is collected by a system of underdrains below. Mechanical filters will purify from 100,000,000 to 200,000,000 gallons of water per acre of filter surface, or from 2,295 to 4,590 gallons per square foot of filter surface, per 24 hours; this rate is from 50 to 100 times that obtained from slow sand filters.

38. In mechanical filtration, a coagulant is generally used to form a sediment layer on top of the filter bed. Coagulants are added to the raw water by means of special apparatus, as will be explained further on.

GRAVITY MECHANICAL FILTERS

39. Continental Filter.—A mechanical filter of the gravity type is shown in Fig. 9. It consists of a wooden tank *a* having its bottom lined with a layer *b* of Portland cement. A system of underdrains *c* rests on the cement. The main underdrains have a large number of branches, each of which is provided with a number of perforated nozzles, as shown. A thick layer *d* of sand serves as the filter bed. The trough *e* is an overflow to carry off the wash water when the filter is being cleaned.

The operation of the filter is as follows: Water, after having been treated by a coagulant, enters the filter through the pipe *f*, its height being automatically regulated by the float *g* inside the tank. The water flows downwards through the sand to the underdrains and is conducted through the pipe *h* to a pure-water tank. The filter bed is cleaned by closing the valves *j*, *k*, and *l*, and alternately forcing water and air through the sand from the underdrains. Water is first admitted through the valve *m* to stir up the bed and thoroughly loosen the dirt, which is floated to the surface by the water and carried over into the trough *e*. The valve *m* is then closed and compressed air admitted through the valve *n* to further agitate the water and aerate the bed. The valve *o* on the waste pipe leading to the sewer is used for flushing the sewer, and is normally kept closed. A regulating device *p*, having a valve and float inside, insures a constant rate of filtration through the sand. When the filter is in operation, the valves *l* and *g* are open and all other valves are closed. When the filter is to be used for the purification of muddy water, separate settling tanks are provided and a coagulant is admitted to the raw water before it enters the settling tank.

Рис. 9

40. Jewell Gravity Filter.—Fig. 10 shows a gravity type of the **Jewell filter.** It differs from the Continental filter principally in the location of the settling chamber and the method of washing the filter bed. The settling basin *a*

Fig. 10

occupies the lower half of a large tank *b*, in which is placed the filter *c*. An annular space *d* is left between the filter and the containing tank for the overflow of water when the filter is being cleaned. Coagulated water enters the settling basin

through the pipe *e*; there the coagulant deposits on the bottom of the tank most of the coarse matter that is in suspension. The clarified water, which still contains sufficient coagulant to form a sediment layer, then rises to the filter through the stand pipe *f*. The strip *g* between the filter and the large tank prevents raw water from rising in the space *d* and overflowing the filter. Air is not used in cleaning this type of filter, but an iron rake *h* is revolved to stir up the sand while water is forced up through the bed from the underdrains. The settling chamber is drained through the valve *k*. The filtered water from the underdrains is drawn off through *i*. The overflow water, while the filter is being washed, is drawn off from *d* through *j*. The gearing, belts, and pulley shown on top of the tank are used to rotate the prongs and thus stir up the sand while it is being washed.

41. Material for Filter Tanks.—Steel tanks have been suggested and used in a few cases in place of wooden tanks. But the freshly coagulated water attacks the steel, even when there is an excess of alkalinity, so that there seems to be some question as to the durability of steel tanks. In the more recent filters, compressed air is used in place of the rake system, and there is therefore no necessity for making the tanks circular. Rectangular masonry tanks have been used, which are much more durable than wooden tanks. They can also be arranged to form part of the building or to carry a masonry vaulting, thereby decreasing the cost of construction and very greatly increasing the durability.

PRESSURE MECHANICAL FILTERS

42. The Jewell Pressure Filter.—There are many kinds of pressure filters on the market, but in a general way the principle of filtration is about the same in all. Fig. 11 shows what is known as the **Jewell pressure filter.** It differs from the gravity type bearing the same name chiefly in being enclosed in a water-tight vessel, in which the water

is forced through the filter bed by the full hydraulic pressure in the supply pipe. The gearing *a* in this and other large filters is operated by power, a belt being attached to the pulley *b*; in small filters, the rake *c* is revolved by hand, a crank being used instead of the pulley. The operation of this filter is as follows: Coagulated raw water enters the

Fig. 11

settling basin at the bottom of the tank through the pipe *d* and the valve *e*. From the settling basin, where most of the matter in suspension is deposited, the coagulated water passes up through the hollow shaft at the center of the tank into the compartment above to the filter bed, through which it is forced to the underdrains below. The pure water, collected by the underdrains, is discharged through the valve *i*

into a pure-water reservoir. When a filter is ready for service again, after having been washed, the filtered water is allowed for a few minutes to run to waste through the valve *h*, until a clear effluent is obtained. The valve *f* is used to wash out mud from the settling chamber into a drain pipe, sewer, or other convenient place of disposal. The valve *g* serves as a by-pass through which unfiltered water can be supplied direct to the fixtures in a building.

COAGULATING APPARATUS

43. The coagulating apparatus for mechanical-filtration plants usually consist of wooden or cement-concrete vats in which the desired coagulating chemicals are mixed, and a measuring pump to feed the coagulants in the proper proportions to the raw water. The mixing vats must be of some non-corrodible material that will not be affected by sulphate of alumina or sulphate of iron; these substances energetically attack and destroy iron and steel. Wood and cement are not appreciably affected by the usual coagulants, and for this reason either material may be used.

The mixing vats for coagulants must be supplied with some means for thoroughly agitating the mixture. Mechanical arms or paddles were formerly used for this purpose, but now the general practice is to use compressed air. All parts of the apparatus that come in contact with the coagulant should be made of some non-corrodible material.

MISCELLANEOUS MATTERS RELATING TO FILTERS

44. Cost of Filters and Filtration.—In recent years, slow sand filters of the best construction in the United States have usually cost, when open, from $30,000 to $40,000 per acre of net filtering area, and about 50 per cent. more when covered with masonry vaulting. Mechanical filters in tubs have cost about $20 per square foot, with one-half as much more for buildings, foundations, and connections. With the newer type of masonry mechanical filters, this cost can be

reduced somewhat. In addition to the cost of filters, the cost of preliminary pumps, reservoirs, conduits, land, etc. must often be taken into account.

The cost of operation of sand filters has ranged from $1 to $5 per million gallons. For reasonably clear lake and river waters, $2 to $3 is a fair estimate. For mechanical filters with sufficient coagulation to give reasonably good bacterial efficiency, the costs of treating similar waters are somewhat higher. For the treatment of very turbid or highly colored waters, the costs are much greater. Generally, the cost of filtration, including interest and sinking-fund charges, has averaged about $10 per million gallons of water filtered; but the cost is considerably greater than this for very turbid waters.

45. Combination of Methods for Various Waters. The slight turbidities of many streams can be entirely removed by sand filtration without any preliminary treatment. Where the raw waters are more turbid, preliminary sedimentation is desirable. Still more turbid waters, such as the waters of many streams in the Middle and Southern American States, cannot be satisfactorily purified in this way, even after preliminary sedimentation. With these waters, coagulation is indispensable. When coagulated, the waters can be successfully filtered by either sand or mechanical filters. Mechanical filters are usually selected on account of their lower cost. For the most turbid waters, full purification involves: (1) sedimentation without chemicals; (2) coagulation; (3) a second sedimentation; and (4) filtration. It may be necessary or advisable to give the water a second or supplementary coagulation just before it goes to the filters. It is possible to remove all the turbidity from even the most turbid waters in this way, producing effluents that are bright and clear.

46. Impurities Removed by the Different Processes.—A small portion of *color*, usually from one-quarter to one-third, is removed by sand filtration. Nearly all the color can be removed by coagulation. The amount of coagulant required to remove color is rather large.

47. Bacteria are removed most completely by sand filtration. Well-designed and operated plants remove over 99 per cent. Bacteria are also removed by mechanical filters when the water is properly coagulated. If the coagulant is omitted, or the amount reduced, the bacterial efficiency at once falls off. At least 1 grain of sulphate of alumina per gallon must be used to secure a reasonably good bacterial efficiency.

48. Tastes and odors are considerably reduced or entirely removed by sand filtration. Mechanical filters are not efficient in this respect, and coagulation is of no advantage. Thorough and continued aeration, however, is the most efficient means of improving such waters, and should be used in connection with filtration.

CHEMICAL PURIFICATION OF WATER

PURIFICATION BY COPPER SULPHATE

49. Copper-Sulphate Treatment.—A method of ridding reservoirs of algæ and other troublesome forms of water vegetation, besides sterilizing the water and thus destroying bacteria, has recently been made public by the Department of Agriculture at Washington, District of Columbia. The method was originated by George T. Moore, Physiologist and Algologist in charge of the Laboratory of Plant Physiology of the Department of Agriculture. The treatment is applied by mixing 1 part of copper sulphate with from 100,000 to 1,000,000 parts of water to be treated. Usually, 1 part of copper sulphate to from 1,000,000 to 2,000,000 parts of water will be found sufficient to destroy the organisms that most frequently cause trouble in water reservoirs.

50. Effect of Copper Sulphate on Fish.—In applying the copper-sulphate treatment to water containing fish, care and judgment must be exercised not to use an amount sufficient to injure the fish. It is perhaps better in treating the water of fish ponds to use a quantity of copper sulphate

that will not injure the fish, even though the original application is insufficient to kill the algæ, and depend on more frequent applications of the weak solution to destroy the objectionable organisms. Fish of different species can stand varying quantities of the chemical. The number of parts of water to 1 part of copper sulphate in dilution that will not injure fish of certain species is given in Table II.

TABLE II

COPPER SULPHATE UNINJURIOUS TO FISH

(*Department of Agriculture*)

Kind of Fish	Parts of Water to 1 Part of Copper Sulphate	Kind of Fish	Parts of Water to 1 Part of Copper Sulphate
Trout . .	7,000,000	Catfish .	2,500,000
Goldfish .	2,000,000	Suckers .	3,000,000
Sunfish .	750,000	Black bass	500,000
Perch . .	1,500,000	Carp . .	3,000,000

51. Method of Applying Copper Sulphate.—The method considered most practicable in introducing copper sulphate into a water supply is outlined by the Department of Agriculture as follows:

"Place the required number of pounds of copper sulphate in a coarse bag—a gunny sack or some equally loose mesh—and attach this to the stern of a rowboat near the surface of the water; row slowly back and forth over the reservoir, on each trip keeping the boat within 10 to 20 feet of the previous path. In this manner, about 100 pounds of copper sulphate can be distributed in 1 hour. By increasing the number of boats, and, in the case of very deep reservoirs, hanging two or three bags to each boat, the treatment of even a large reservoir may be accomplished in from 4 to 6 hours. It is necessary, of course, to reduce as much as possible the time required for applying the copper, so that for immense supplies with a capacity of several billion gallons it would probably be desirable to use a launch, carrying long projecting

spars to which could be attached bags each containing several hundred pounds of copper sulphate.

"The substitution of wire netting for the gunny-sack bag allows a more rapid solution of the sulphate, and the time required for the introduction of the salt may thus be considerably reduced."

The copper-sulphate treatment will not only destroy water organisms of the algæ species, but will also destroy most of the bacteria found in water. Nevertheless, the Department of Agriculture does not recommend the copper-sulphate process instead of filtration for the sterilization of water supplies, but only as an additional precaution to safeguard the public in times of threatened danger from bacteria.

REMOVAL OF IRON FROM GROUND WATER

52. Removal of Ferrous Carbonate.—Water that contains iron in the form of ferrous carbonate can be rendered fit for use by **aeration**; that is, by exposing it to the action of air. The effect of this action is to transform the carbonate, which is soluble, into ferric oxide, which is insoluble and can be subsequently removed by filtration. Conducting the water for a considerable distance through open, wide, shallow flumes or channels will expose it to the action of the air, which will precipitate the iron from the carbonate. Passing the water through coke filters not only reduces the iron to the insoluble form, but removes it by filtering it off. The usual method of aerating water for the removal of ferrous carbonate is to allow it to flow over weirs, or discharge into a reservoir or tank through the upturned end of a pipe.

53. Removal of Ferrous Sulphate.—Iron in the form of ferrous sulphate is more costly to remove than iron in the form of ferrous carbonate. The removal is effected by the use of some reagent. Lime is generally used for this purpose. The effect of the lime is to reduce the iron to an insoluble form, which is afterwards removed by filtration. Adding lime to water increases not only the cost of operating a plant but also the hardness of the water.

SOFTENING OF HARD WATER

PRINCIPLES OF WATER SOFTENING

54. Softening Temporarily Hard Water.—Temporary hardness can be removed by adding lime to the water to be treated. When lime is added to water containing carbonate of lime or magnesia, it acts on the carbonate of lime, which is in solution in the form of bicarbonates, releasing the extra carbonic acid required to form the bicarbonates, and precipitates the carbonates of lime or magnesia, which are insoluble. The carbonates are then removed by passing the treated water through a filter.

55. The amount of lime required depends on the degree of hardness of the water. Ordinarily, 1 pound of lime will precipitate about 2 pounds of carbonate of lime. As each

TABLE III

CHEMICALS THAT WILL SOFTEN TEMPORARILY HARD WATER

Substance	Quantity Required to Precipitate 1 Pound of Carbonate of Lime or Magnesia Pounds
Lime	.56
Trisodium phosphate	2.18
Caustic soda	.80
Barium hydrate	3.75
Tannic extract	11.92
Sugar	2.28

degree of hardness in water is equivalent to 1 grain of carbonate of lime to 1 imperial gallon, then, when the hardness of a water is known, the amount of lime required to remove the hardness can be readily calculated.

56. Much of the temporary hardness of a water can be removed by boiling the water in an open vessel. Heating

the water to the boiling point drives off some of the carbonic acid and allows the remaining carbonate of lime to precipitate in the vessel. This method of ridding water of hardness is extensively used for purifying feedwater for steam boilers. Substances that will precipitate carbonate of lime can be found in Table III.

57. Softening Permanently Hard Water.—Soda ash is generally used as a reagent to soften permanently hard water. When soda ash is added to hard water, it acts on the sulphates of lime and magnesia present in solution, decomposes them, and forms insoluble compounds that are precipitated in the vessel.

58. Other Substances That May Be Used for the Softening of Hard Waters.—Other reagents, besides lime and soda ash, can be used to soften hard waters, but

TABLE IV

CHEMICALS THAT WILL SOFTEN PERMANENTLY HARD WATER

Substance	Quantity Required to Precipitate 1 Pound of Sulphate or Chloride of Lime Pounds
Soda ash	.85
Sal soda	1.94
Barium chloride	1.53
Tannic extract	8.76
Sugar	1.68
Trisodium phosphate	1.60

these two substances are generally preferred, owing to their cheapness and to the fact that they are readily obtained in any market. Chemicals that will soften permanently hard waters can be found in Table IV.

WATER-SOFTENING APPARATUS

59. Intermittent Softening Apparatus.—Water supplies that are obtained from rivers vary in hardness, from time to time, according to the amount of rainfall, the hardness being greatest during periods of drought. Such waters are best treated by the **intermittent process,** in which a quantity of water in a tank or reservoir, after being analyzed to determine its hardness, is treated with the required amount of chemical to precipitate the carbonates or sulphates.

60. In treating water by the intermittent process, a tank is filled with water, the hardness of the water determined, and the proper amount of chemical added; the water is then well agitated to mix it thoroughly with the chemical. A period of rest is then allowed for the insoluble matter to settle to the bottom of the tank. The water is then drawn off and stored in a soft-water tank ready for use. It is desirable with this process to have two precipitation tanks, so that water can be treated in one while being drawn from the other.

61. Continuous-Flow Softening Apparatus.—Water in which the degree of hardness remains fairly constant can be treated by the **continuous-flow softening process.** In this method, the chemical reagents are automatically fed to the raw water in proportion to the amount of softened water used. The water is then passed through perforated plates and layers of filtering material, which intercept the precipitated matter.

SEWAGE PURIFICATION AND DISPOSAL

SEWAGE ANALYSIS

COMPOSITION OF SEWAGE

1. Organic and Inorganic Matter.—For the purposes of sanitary science, matter is divided into two general classes; namely, *organic* and *inorganic* matter.

Organic matter is the substance of which living bodies, whether animal or vegetable, are composed. It is very unstable, and, after the organism dies, gradually passes into inorganic compounds through a process of chemical decomposition.

Inorganic matter is the name applied to all matter existing outside of living organism or that is not necessarily a constituent of living beings.

The simple substances of which organic matter is composed are found also in inorganic matter, but differently combined.

2. Effects of Organic and Inorganic Matter on Sewage.—Both organic and inorganic matter are contained in sewage, but, as a rule, the inorganic matter is of little sanitary importance, since it possesses no disagreeable nor dangerous properties, and is chiefly objectionable on account of the volume it contributes to the amount of sludge or waste that must be removed from the purifying tanks. The inorganic matter amounts to about 1 part in 5,000 parts of the sewage, or, for a city of 100,000, about 20 tons a day, which

must be handled and disposed of. The organic matter that is objectionable in sewage seldom amounts to more than 1 part per 1,000 parts of sewage in cities with an adequate water supply. Yet, it is this small amount of organic matter that, by reason of the putrefactive changes through which it passes, and of the lower forms of life that may be associated with it, gives sewage its disease-producing power, as well as its offensive odors.

3. Principal Constituents of Sewage.—The matter that enters into the composition, and has to be dealt with in the purification, of sewage may be classed as follows:

1. *Solid Feces.*—The solid feces consist of nitrogenous partly digested matter and vegetable residues of food. The former are easily liquefied, but the latter dissolve slowly.

2. *Urine.*—Urine is the main source of ammonia, from the fermentation of urea. There is, by weight, about thirteen

TABLE I

WEIGHT, IN POUNDS, OF THE SOLID AND LIQUID EXCREMENT OF A SINGLE PERSON FOR 1 YEAR

Composition	Feces	Urine
Total weight	72.70	941.00
Dry substance	23.24	34.50
Organic matter	20.70	22.60
Mineral matter	2.50	12.00
Nitrogen	1.20	10.80
Phosphoric acid	.70	1.93
Potash	.24	2.01
Carbon	10.00	12.00

times as much urine excreted daily as there is solid feces, and there is as much organic and inorganic matter contained in the urine as there is in the solid feces, and perhaps more. It should not be supposed, therefore, that because urine is liquid and feces are solid, the latter are more objectionable;

for urine is just as disagreeable when undergoing putrefaction, and, when voided by diseased persons, is quite as likely to be charged with pathogenic bacteria. The weights and composition of feces and urine are given in Table I.

3. *Household Waste.*—The larger solids of household waste are disposed of as garbage, but wash water, vegetable refuse, fragments of food, and wastes from laundry tubs, and kitchen, scullery, and pantry sinks are discharged into the house drain, where they contribute to the volume of sewage. This kind of household waste forms a large proportion of the sewage from any city. In bulk, it amounts to about one-half the daily flow, and the dish water and waste from scullery sinks contain so much grease that they contribute that part of the sewage which is, perhaps, the most difficult to treat successfully.

4. *Street Washings.*—With street washings may properly be classed the drainage from cultivated and uncultivated areas that drain into the sewer. The matter reaching the sewer from these sources generally consists of cellulose material—such as hay, leaves, paper, and horse droppings—together with more or less mineral matter, both in suspension and in solution. At times, also, finely disseminated particles of clay, sand, and loam are washed from the streets into the sewer. Cellulose material is difficult to reduce to its original compounds, and if it is discharged without reduction on filters, it clogs the surface of the filter beds, thus interfering with their operation. The clay and sand are even more objectionable, since they fill up the voids in filter beds, thus excluding the sewage.

5. *Grit and Detritus.*—Sand, gravel, and large particles of floating matter carried by storm water from street surfaces into combined sewerage systems are known as **detritus.** Such substances are of but slight importance in the purification of sewage, as they are screened out or intercepted by detritus chambers, and are not allowed to reach the purification works.

6. *Manufacturing Wastes.*—Many industrial establishments produce wastes of such nature that when discharged

in large quantities into the sewers they may interfere greatly with the process of sewage purification. It is customary, under such conditions, for a city to require that the manufacturing wastes, before being discharged into the public sewers, be treated on the premises by the proprietors.

4. Hourly Variations in Character of Sewage. There is a great variation in the quantity and in the chemical composition of sewage as it is ordinarily discharged at different hours during the day and night, and this variation has a material influence on the capacity and arrangement of a purification plant. During the early hours of the day (about 2 A. M.), the sewage is the least in volume, and is also the weakest chemically. From this time on, it increases in strength and volume during the day, and between about 8 A. M. and 2 P. M. the flow is at a maximum. After 2 P. M., the quantity of sewage becomes less, but the strength increases until about 8 P. M., when the sewage is the strongest.

The daily variations in the strength of sewage do not affect the design of a purification plant so much as do the variations in volume, but they affect very greatly the method of treatment. The plant must be designed to accommodate the maximum flow, and variations in either strength or volume are commonly provided for by a continual change of treatment.

5. Progressive Changes in Sewage.—Sewage is an unstable product that is constantly passing through successive transformations tending toward the complete breaking up and oxidation of the organic matter. So rapid are these natural changes that samples of sewage taken at various points along an out-fall sewer of considerable length show a distinct progression in some of these transformations. Chemical analyses of sewage have to do principally with the organic matter it contains, and are directed toward tracing the quantity present, its character, and the transformations it undergoes in the process of purification. Many of the phenomena that occur in the process of purification are intricate and

variable and not thoroughly understood, but some general principles have been formulated that it is necessary to understand in order to properly design and operate sewage-purification plants.

INTERPRETATION OF SEWAGE ANALYSES

6. Data Obtained by Analyses.—Analyses of sewage for organic matter generally show the total dry solids, the proportion of these that are combustible (generally referred to as "loss on ignition"), and the proportion of ash remaining (referred to as "fixed residue"), the amount of free ammonia, albuminoid ammonia (sometimes given as "organic nitrogen"), chlorine, the amount of oxygen required to oxidize the organic matter (called "oxygen consumed" or "oxygen absorbed"), and the number of bacteria per cubic centimeter.

7. Meaning of Analysis.—The meaning of a chemical analysis, giving values of the quantities just named, is suggestive rather than positive. The total solids separated by filtration are burned, and the loss in weight is assumed to

TABLE II

RESULTS OF TREATMENT OF VARIOUS EFFLUENTS

Source of Effluent	Distillate Method		Oxygen-Lost Method	
	Parts per 100,000	Purification, in Per Cent.	Parts per 100,000	Purification, in Per Cent.
Open settling tank	.31		7.00	
First filter	.15	50	2.21	68
Second filter . . .	.06	81	.69	90

bear some ratio to the total organic matter present. The sewage is evaporated, the gases condensed, and the distillate tested to give the amount of organic matter. Or, the sewage is mixed with a compound containing a large proportion of oxygen, and the amount of oxygen taken by the sewage from that compound measures the amount of organic matter. The

TABLE III

ANALYSES OF SEWAGE AND OF WATER SUPPLIES

(*Parts in 100,000*)

	Sample Analyzed	Dry Solids: Combustible	Dry Solids: Ash	Dry Solids: Total	Ammonia: Free	Ammonia: Albuminoid	Ammonia: Total	Chlorine	Oxygen Consumed	Bacteria per Cubic Centimeter
Sewage	1. Sewage	24.5	32.7	57.2	1.7	.63	2.33	8.3	4.5	2,000,000
	2. Sewage	21.0	16.0	37.0	2.7	.62	3.32	3.0	6.3	
	3. Sewage, average in sixteen English towns			116.9				10.6		
	4. Sewage, London, England	49.2	74.3	123.5	4.6	.55	5.15	15.2		
	5. Sewage	32.0	58.0	90.0	.9	1.60	2.50	8.4	5.6	900,000
	6. Sewage from glucose factory	366.0	284.0	650.0	4.4	27.20	31.60	3.6	127.0	1,800,000
	7. Sewage from packing house	122.0	688.0	810.0	2.3	78.70	81.00		115.0	3,540,000
Water Supplies	Hemlock Lake	3.2	6.2	9.40	.000	.008	.008	.20		
	Merrimac River	1.7	2.3	4.00	.006	.020	.026	.20	.63	
	Lake Superior			5.40	.003	.002	.005	.10	.10	
	Mountain Spring			22.80	.004	.005	.009	.40		
	Lake	3.5	10.0	13.50	.002	.016	.018	.11	.40	
	Ice			.32	.004	.003	.007	.00	.02	

method by loss of weight, while still given in analyses, is probably the least accurate of the three processes. The other two are both used, and it is common—for example, in giving the efficiency of a filter—to use both processes for comparison. Table II shows the statement made for Manchester, England, as the average purification for a 10-week period.

In Table III are shown the results of the analyses of different samples of sewage, and, for purposes of comparison, of various water supplies that are considered fit for domestic use.

8. Sources of Organic Matter.—The organic matter in sewage is derived from various sources, and is not all equally harmful. When discarded by animal life, and particularly by human life, it is the most dangerous, for it may contain germs of disease; and, though chemical analysis may show a marked degree of purification, an effluent from sewage that contains organic matter derived from this source is more dangerous than one in which the organic matter is derived from plant life, and particularly from vegetable matter that has not been used as food.

The source from which organic matter in sewage is derived is to some extent indicated by chemical analysis. For instance, it is known that analysis No. 6 in Table III is the washings of vegetable matter in the glucose process; that analysis No. 7 is the waste from a slaughter house, and contains much animal refuse; and that analysis No. 4 is an average sewage of London, containing both animal and vegetable matter. Comparing the results of the analyses, it will be seen that the ratio of albuminoid ammonia to the total combustible solids in each case is as follows:

In No. 6 (mainly vegetable matter), $\frac{27.2}{366}$, or .075.

In No. 7 (largely animal waste), $\frac{78.7}{122}$, or .65.

In No. 4 (London sewage), $\frac{.55}{49.2}$, or .011.

It will be observed that the ratio is greatest in the case of sewage consisting mainly of animal waste. Taking up the

oxygen consumed in each case, it will be found that there is relatively and actually more oxygen required in the case of the glucose waste, which is of vegetable origin.

Thus, the combustible solids, the albuminoid ammonia, and the oxygen consumed all indicate the presence of organic matter, while their relative amounts serve as indications of the particular character or derivation of the organic matter.

9. Total Dry Solids.—The dry solids in sewage indicate, in a measure, the concentration of the sewage, but may vary widely in the proportion of organic and mineral matters that they contain. Some of the solids are in suspension and some are dissolved in the sewage. The proportion of total dry solids is generally reduced in the process of purification. In some chemical processes, however, the effluent, although containing less organic solids than the sewage, contains a greater proportion of total solids; this is due to the formation of inorganic compounds from combinations of the chemicals added with the substances originally in solution in the sewage.

10. Combustion: Loss on Ignition.—Combustion measures the organic matter in the sewage, but does not serve to distinguish between that of animal origin and that of vegetable origin. The proportion of combustible matter is greatly reduced in all purification processes.

11. Fixed Residue: Ash.—The residue or ash left after combustion is a stable substance that may be derived largely from earthy matters, and is of importance as affecting the amount of sludge that it may be necessary to handle in a purification plant. In chemical-purification processes, the effluent usually contains a greater proportion of ash than the sewage, for the reason that part of the chemicals used as precipitants remain in solution or suspension and pass off in the effluent. In biological processes, the effluent contains a much smaller proportion of fixed residue than the sewage. If the result of the ignition is to turn the residue black, and at the same time to emit a strong earthy smell, the organic matter is assumed to be vegetable. If, however, the residue turns or stays white, the presence of mineral solids is indicated.

12. Free Ammonia.—Free ammonia is derived mainly from organic nitrogen, and is produced in one of the intermediate stages of the dissolution of organic matter. As the process of purification continues, ammonia finally changes into the harmless nitrates. Ammonia marks the first step in the process of decomposition, and, while the analysis does not show its origin, its presence indicates contamination, though not necessarily sewage contamination, as the ammonia may come from other sources, such as decaying leaves and swampy soils.

13. Albuminoid Ammonia: Organic Nitrogen. Albuminoid ammonia does not exist ready formed in sewage, but is derived in the analysis by decomposing the organic nitrogenous substances. The quantity of albuminoid ammonia is taken as a measure of the amount of organic nitrogen present. In the process of purification, the albuminoid ammonia or organic nitrogen is broken up and changed, like the free ammonia, into nitrates. The determination of this substance is very important, as it indicates the presence of animal wastes, the most dangerous constituents of sewage. The proportion should, therefore, be reduced as much as possible in any treatment adopted.

14. Chlorine.—Chlorine is a most persistent constituent of urine, both human and animal. Consequently, the quantity of chlorine present is an indication of the concentration of sewage and of the extent to which natural waters have been polluted by sewage. Purification of sewage, whether natural or artificial, has generally no marked effect on the amount of chlorine, and, therefore, sewage contamination of a stream may be detected by an excess of chlorine when all other signs of organic matter have passed away.

15. Oxygen Consumed.—The ultimate result sought in the purification of sewage, so far as chemical analysis is an indication of purity, is the complete oxidation of the organic matter; and so the amount of oxygen necessary to accomplish this is an indication of the amount of impurities in the sewage. The quantity necessary for this purpose is

determined experimentally by adding to a definite amount of sewage, compounds of known composition, which will lose the oxygen necessary to oxidize the organic matter. The process mainly measures the carbonaceous matters in sewage, which are not the most dangerous.

16. Nitrites and Nitrates.—Nitrites and nitrates are formed from nitrogen derived from the breaking up of organic substances, and recombined with oxygen. The complete reduction of the organic compounds and the combination of their nitrogen with oxygen in the form of nitrates indicates the final stage of purification; hence, the progress of the purification of sewage is measured by the increase in

TABLE IV

LOSS OF ORGANIC MATTER IN PURIFICATION PLANT

(*Parts in 100,000*)

Sample Analyzed	Organic		Intermediate	Inorganic
	Free Ammonia	Albuminoid Ammonia	Nitrites	Nitrates
Sewage	1.989	.843	.0001	.000
Purified effluent	.004	.010	.0003	1.880

nitrates. Table IV shows the reduction of organic nitrogen and the increase of nitrates in the process of purification in a sand filter.

BACTERIAL COMPOSITION OF SEWAGE

17. Bacteria.—As stated in *Purification of Water*, Part 2, **bacteria** are microscopic organisms belonging to the vegetable kingdom. They are closely allied to the fungi, yeasts, and molds. According to their mode of living, bacteria are classified as *aerobic*, *anaerobic*, and *facultative*. **Aerobic** bacteria live and multiply only in the presence of air or of free oxygen. **Anaerobic** bacteria thrive only in the absence of oxygen. **Facultative** bacteria can live and thrive either in the presence or in the absence of oxygen.

18. Bacteria are further divided into *saprophytic* and *parasitic*. **Saprophytic,** or refuse-eating, bacteria obtain their nutriment from dead organic matter, exist independently of a living body, and are generally harmless to living organisms. It is the saprophytic bacteria that bring about the changes that reduce organic matter to harmless compounds; hence, this class of bacteria is of the greatest aid and importance in the process of sewage purification. **Parasitic** bacteria live on or in some other organism, from which they derive nourishment. They cannot thrive independently of a living body, and are of special importance in sewage purification, because some varieties of them are the cause of many diseases.

19. Parasitic bacteria are divided into *pathogenic* and *non-pathogenic*. **Pathogenic** bacteria are those that produce disease in man or other animals. **Non-pathogenic** bacteria are those that, although living as parasites, do not produce disease in the bodies in which they live.

20. Infectious Diseases.—When pathogenic bacteria penetrate into living organisms and grow and multiply therein, they cause special diseases, the nature of which depends on the species of the bacteria causing them. Diseases caused by bacteria are called **infectious, or zymotic, diseases.**

21. Sources of Infection.—Pathogenic bacteria may be taken into the system in food, in drinking water, with the air breathed, or through abrasions in the skin. The sources of infection most important in studying the subject of sewage purification and disposal are food and drink, as these sources are more liable than others to pollution from sewage.

22. Presence of Bacteria.—Bacteria are always naturally present wherever organic matter exists, and their number is nearly proportional to the amount of organic matter. The efficiency of water-filtration plants is generally expressed in terms of the number of bacteria before and after filtration. Bacteria are present in air, in the soil, in water, and

in all organic matter that may exist in any of these elements. Spring water and deep-well water may be free from bacteria, but ordinary surface water, whether contaminated by sewage or not, contains a great many of them.

23. The number of bacteria present in any water is expressed as the number per cubic centimeter (a cubic centimeter is equal to about a thimbleful). The presence of more than 100 bacteria per cubic centimeter is considered to make the water unfit for use. As water becomes contaminated with organic matter, the number of bacteria increases in proportion to the amount of organic matter, becoming 3 or 4 million per cubic centimeter in badly polluted streams. Since the effect of the presence of saprophytic bacteria is to oxidize organic matter, which in turn is the desired end of all sewage-purification processes, it follows that a large number of such bacteria present in sewage during its treatment is most desirable and beneficial. It also follows that any process that introduces chemicals or methods that attack these bacteria is fundamentally wrong in principle.

24. Pathogenic Bacteria.—The good effects of saprophytic bacteria must be fully appreciated, and at the same time the bad effects of pathogenic bacteria must not be forgotten. These latter are not found except in water polluted by animal waste, though the number of pathogenic bacteria is supposed to bear an approximately constant ratio to the total number of bacteria. The detection of pathogenic bacteria is still a difficult matter, even with the present refinements of bacterial science.

25. Method of Bacterial Analysis.—Analysis of sewage for bacteria consists: (1) in determining the number per cubic centimeter, and (2) in applying certain tests to differentiate various species. Both processes are somewhat uncertain, and the results must be interpreted with judgment. It is possible, by mixing the water to be tested with liquid gelatine, to fix the individual bacteria in the gelatine when it is cooled in a thin layer on a glass plate.

In a few days, each original develops into several thousand, which then appear as a white spot, or a colony. It is assumed that each colony represents one bacterium in the sewage.

DISSEMINATION OF INFECTIOUS DISEASES

26. Infection From Men and Animals.—Typhoid fever and Asiatic cholera may be taken as types of epidemic infectious diseases. It has been found that every case of typhoid fever arises from an antecedent case, and in this way only. For a limited time, and under conditions particularly favorable, pathogenic bacteria may multiply outside the human body. It may be safely said, however, that, for the most part, man and other animals are the main if not the only immediate sources of infection.

27. Diarrheal Diseases.—An important class of infectious maladies, known as **diarrheal diseases**, are directly attributed to infection from the bowel discharges. Typhoid fever, Asiatic cholera, dysentery, and diarrhea are the most important diseases of this class. Bowel discharges may readily be mingled with and infect almost any substance in the environment. It is when they are mingled with a stream that they are particularly dangerous. The disease germs contained in diarrheal discharges may be disseminated by air, dirt, dust, sewage, water, ice, milk, vegetables, insects, and in other ways.

28. Typhoid Fever.—Typhoid fever may be taken as a typical diarrheal, water-borne disease. It has been established that the direct cause of typhoid fever is a microscopic organism (*Bacillus typhosus*) that attacks the intestine. The vitality of this organism is remarkable. It can thrive both in the absence and in the presence of oxygen. It can withstand a degree of heat near boiling. It lives frozen in blocks of ice for indefinite periods. It can live and multiply to some extent in water, and especially in sewage-polluted water. It has been found alive after 150 days in distilled water, 90 days in Lake-Michigan water at Chicago, 75 days in the

water of the Thames at London, and 60 days in the Ohio-River water at Cincinnati.

29. Polluted Water Supply.—Water is one of the most common vehicles of disease, and by far the greater number of infectious diseases can be traced to an infected water supply. It was formerly believed that the water of a flowing stream purifies itself, and that the dilution of a foul water to such an extent that chemical analysis could not detect sewage contamination rendered such water innocuous. Later investigations, however, disproved these theories, especially as regards the destruction of pathogenic bacteria. Dilution, time, and sedimentation are the factors that contribute most to the purification of flowing streams. These factors, however, cannot be depended on, except in a very slight degree, to destroy the germs of infectious diseases with which water has been infected. This is shown by the statistics of numerous cities that have taken their water supply from streams subject to pollution above the point of intake.

An outbreak of typhoid fever occurred at Plymouth, Pennsylvania, during which 1,000 to 1,200 cases developed between April 12 and April 22, 1885, in a population of 8,000. The outbreak was traced to infection from the dejections of a single patient, which were thrown out during the winter months on the banks of a stream discharging into a reservoir that furnished the water to the village. The typhoid germs retained their vitality until the spring thaw, when they were washed into the stream and thence found their way into the public water supply. As time went on, the current carried the typhoid germs below the waterworks intake, or, the conditions being unfavorable to their continued life and development, they lost their vitality, and the epidemic ceased.

30. Ice as a Vehicle of Infection.—Water, in freezing, rids itself of a great proportion of the impurities that it contains, and, while bacteria may survive for considerable periods when frozen in ice, only a very small percentage of the number originally contained in the water survive for any great length of time, and the activity of these few survivors

is materially lessened by the unfavorable temperature to which they are subjected. Infection has, in some instances and under favorable conditions, been spread by the use of ice. However, no considerable amount of disease has ever been traced to this source, and, as a vehicle of disease, ice is far less dangerous than water to the public health.

31. Pollution and Infection.—There is a marked difference between general *pollution* of a water supply and specific *infection*. A limited degree of pollution with uninfected sewage, containing no specific disease germs, may do little harm, although the drinking of water so polluted undoubtedly tends to lower the general health. On the other hand, a water that is clear and apparently unpolluted may contain the germs of a specific disease, and be, therefore, dangerous. This is an important matter to be kept in mind in considering the subject of sewage purification. It is not exaggerating the danger to say that water into which sewage has been discharged, without purification, is thereafter unsafe for drinking purposes. The danger is modified by the presence of pools and other opportunities for sedimentation. But the danger exists even beyond the pools, and one typhoid-fever patient whose discharges have been thrown into a stream may communicate the disease to thousands drinking the water. No such possibility should be allowed, and where the water of a stream is used for drinking, sewage should be purified before it is admitted to the stream. The number of bacteria present in sewage, and particularly the extent to which they are reduced in the process of purification, is an important indication. Only a small proportion of the bacteria in sewage are disease-producing, but it is presumed that any process of purification that reduces the total number will reduce the pathogenic bacteria in as great and probably in a greater proportion.

TESTS OF SEWAGE EFFLUENT

32. The Incubation Test.—The incubation test consists in exposing the effluent—either clear or mixed with other waters, as the case may require—in an incubator or warmed chamber, kept at such a temperature that putrefaction may be hastened if there is any tendency for it to occur. This test is often applied to determine the stability of sewage effluents, or whether they will still further putrefy when mixed with the water of a stream.

33. The Fish Test.—Fish are greatly affected by the impurities in a sewage effluent, and their actions indicate the conditions of the effluent so far as its contained oxygen is concerned. Fish cannot live long in sewage-polluted water from which the oxygen is exhausted, and the presence of little fish in a stream into which sewage has been discharged after purification is an indication that the purification has been successful.

METHODS OF PURIFICATION AND DISPOSAL

DISCHARGE OF SEWAGE INTO STREAMS AND LAKES

34. The Natural Method.—Most cities and villages have grown up along some watercourse or water front toward which the sewage must be conveyed, if it is discharged by gravity, and into which it is very natural and convenient to discharge it. This is almost universally the first method of disposal and one that is persistently adhered to by a community until the interests of other communities interfere to protect the purity of the stream, or until the pollution of the water front becomes a serious nuisance. While a marked degree of self-purification is effected in a body of water that is contaminated in this way, it is the conclusion of sanitarians that crude sewage never should be

discharged into any body of water used as a water supply at any point within the influence of the sewage.

The degree of pollution that may be permitted, and the extent to which a stream or body of water recovers its purity, are important considerations. All streams are not to be considered in the same class. Many of them, particularly along their lower stretches, are not suitable for water supplies, and may well be given over to a degree of pollution that should not be permitted in others. No nuisance will generally be created so long as the dilution is sufficient to prevent putrefaction. The water being exposed to the air, the greater its comparative volume and the more rapid its current, the more rapid and thorough will the purification be. When the amount of sewage is small in comparison with the volume of water, and, especially, where the distances between towns are great and the stream has a swift current, this method of sewage disposal may be employed with reasonable safety.

35. Purification by Dilution.—Simple dilution may be classed as purification; but dilution alone does not fully measure the purification that follows when sewage is discharged into a stream or other body of water. There are natural agencies active in purification under these conditions, which carry the process much farther than simple dilution. Some of these purifying agencies are: sedimentation, animal life, bacteria, and oxidation. Each of these factors plays an important part, not only in the purification of crude sewage, but in the further purification of effluents from certain purification processes, which must be discharged into streams, and which, by the partial treatment they have received, are rendered particularly susceptible to these agencies.

36. Oxidation in the Illinois and Michigan Canal. Water that is not too badly polluted can be purified by oxidation. An example of such purification is afforded by the Illinois and Michigan Canal. In this canal, the amount of sewage entering at Bridgeport was about one-seventh of the water flowing, the distance from Bridgeport to Lockport was

about 29 miles, the current was sufficient to prevent sedimentation, and the condition of the channel was less favorable to microscopic life than in ordinary streams. The changes that took place were, therefore, due almost entirely to oxidation. They are shown in Table V, the figures given representing parts in 100,000.

TABLE V

PURIFICATION OF SEWAGE BY OXIDATION

(*Illinois and Michigan Canal*)

Place	Total Solids	Suspended Solids	Chlorine	Free Ammonia	Albuminoid Ammonia	Oxygen Consumed
Bridgeport .	47.12	12.92	4.68	1.23	.26	2.31
Lockport . .	43.12	6.98	4.61	1.08	.20	1.62

The analysis shows the oxidation of 30 per cent. of the oxidizable matter. The proportion of chlorine was not materially changed, which is an indication that there was little dilution between these points, and that the purification was mainly due to oxidation. The oxidation shown was not sufficient, however, to remove the nuisance that existed at the upper end, and, even after the 30-per-cent. reduction, the water was foul, and had a most unpleasant odor.

37. Effect of Dilution on Bacteria.—Dilution has an unfavorable effect on the vitality and development of pathogenic bacteria, for it surrounds them with a medium in which their food supplies are limited. Sedimentation is also an important factor in removing bacteria. These agencies—sedimentation and limited food supply—are active in lakes, and to them, mainly, must be due the destruction of disease germs that is indicated by the surprising immunity from water-borne diseases in many lake cities that obtain their water supplies from the same body of water into which their sewage is discharged.

CHEMICAL PRECIPITATION

38. When the volume of sewage is a great proportion of the flow of the stream, the disposal by dilution is not satisfactory. The water becomes turbid and black. Bubbles of gas continually rise and break from the surface. Deposits of organic matter occur on the bed of the stream and against the banks where the current impinges. Decomposition takes place without enough oxygen for oxidation, and the stream is one mass of putrid filth, which pollutes the air for a distance of hundreds of yards. Under such conditions, the process of simple sedimentation has been practiced in a few cases. It has seemed more feasible, however, to add to the sewage certain chemicals, by whose agency a precipitate is formed that settles rapidly and carries down with it nearly all the suspended matter and also a portion of the matter in solution. This process is called **chemical precipitation.**

39. Requirements of Chemical Precipitants.—The substances generally used as precipitants are lime and the salts of aluminum and iron. A precipitating agent should possess the following requisites: (1) it should be cheap and abundant; (2) it should cause rapid subsidence of the precipitate formed; (3) it should not be poisonous nor produce poisonous compounds; (4) it should not color the solution; (5) the precipitated sludge should part with its moisture readily; (6) since the cost of treating the precipitated sludge is an important factor, the precipitation should not increase the quantity of sludge beyond what is necessary to effect the purification.

The kind and quality of chemicals most suitable to be used in a given case depend on the character of the sewage and the cost of various chemicals.

40. Purification of Manufacturing Wastes.—Chemical purification is particularly adapted to the treatment of manufacturing wastes, which are the result of chemical processes, and to the treatment of sewage that is in part

made up of such wastes and is less susceptible than ordinary sewage to biological treatment. Chemical precipitation may serve as an intermediate or preliminary process in the treatment of manufacturing wastes before they are mingled with the ordinary sewage that is to be treated biologically, provided the chemicals are not added in such quantity as to destroy all bacterial life.

41. Arrangement of Precipitation Plants.—In chemical-precipitation plants, the operation and arrangement are generally as follows: First, the precipitant is dissolved in a mixing tank, and then the solution is discharged into the sewage in a small stream. The sewage is roughly tested by chemical reagents, at frequent intervals, and the amount of precipitant admitted is proportioned to the amount of sewage and its content of organic matter, so that there may be no waste of precipitant and no unnecessary increase in the amount of it that is thrown down in the sludge over that required to accomplish the purpose. It is important that the precipitant be thoroughly mixed with the sewage, and, in order to accomplish this, the sewage, after the precipitant is applied, is passed over cascades or through tortuous channels in which are set baffle walls that thoroughly incorporate the precipitant with the sewage. The same end is accomplished in some cases by mechanical mixers driven by power.

The mixture of sewage and precipitant is then allowed to flow into settling tanks, in which it is held for a certain time, until the sludge is precipitated. The clarified sewage is then drawn off, and the separated sludge is pumped to filter presses, where the greater portion of the water is extracted. The sludge has little manurial value and is often used for filling in low ground.

A series of tanks is generally provided, so that, while the sewage is standing in one, some of them may be filling, and the sludge may be draining from the others.

42. Continuous Process of Precipitation.—Sometimes, the process of precipitation is a continuous one, the effluent being constantly drawn from the tank at a certain

level, and the sludge, which settles to the bottom, being drawn off at intervals through a sludge valve at the bottom of the tank. A plant of this kind was used for treating the sewage from the World's Columbian Exposition at Chicago. The sludge was first put through a press to remove as much water as possible, and then was burned in a garbage crematory. The cost of precipitants was about $8 per million gallons of sewage.

TABLE VI

RESULTS OF CHEMICAL PRECIPITATION

Chemicals per 1,000,000 Gallons	Settled Hours	Per Cent. of Organic Matter Removed	
		Loss on Ignition	Albuminoid Ammonia
1,000 pounds of lime . . .	20	32	62
2,000 pounds of lime . . .	3	38	39
2,000 pounds of lime . . .	4	56	52
2,000 pounds of lime . . .	4	57	43
1,600 pounds of lime . . .	27	41	25
2,100 pounds of lime . . .	20	18	32
500 pounds of alum . . .	1½	13	35
500 pounds of alum and 800 pounds of lime . . .	1½	43	38
500 pounds of alum and 800 pounds of lime . . .	1	20	26
500 pounds of copperas and 600 pounds of lime . . .	1	60	22

43. Results Obtained by Chemical Precipitation. Tables VI and VII show the results obtained by chemical precipitation at the experimental station of the Massachusetts State Board of Health.

44. Purification Due to Settling Without Chemicals.—These tables show that more than one-half of the organic matters removed, as indicated by the loss on

ignition and by the relative amounts of albuminoid ammonia, might have been removed by the simple settling of the sewage without the addition of any chemicals.

TABLE VII

COST AND EFFICIENCY OF CHEMICALS

(*Sewage Allowed to Settle 1 Hour*)

Sample	Cost for Chemicals per Inhabitant Annually	Per Cent. Loss on Ignition Removed	Per Cent. of Albuminoid Ammonia Removed
Sewage after settling . . .	.00	30	26
Effluent with 700 pounds of lime	.11	39	33
Effluent with 500 pounds of alum	.23	27	40
Effluent with 500 pounds of alum and 700 pounds of lime	.34	37	48
Effluent with 500 pounds of copperas	.09	36	21
Effluent with 500 pounds of copperas and 700 pounds of lime	.20	48	50
Effluent with 120 pounds of ferric oxide	.13	64	33
Effluent with 120 pounds of ferric oxide and 700 pounds of lime	.24	57	51

45. Results With Theoretical Quantity of Precipitant.—In an experiment made by the Massachusetts State Board of Health, in which the amount of lime used as a precipitant was adjusted to the amount of carbonic acid in the sewage, all the suspended organic matter—20 per cent. of the soluble albuminoid ammonia, 15 per cent. of the

soluble loss on ignition, and 97 per cent. of the bacteria—was removed at a cost for chemicals of $7.31 per 1,000,000 gallons, or 27 cents per inhabitant, annually. The amount of lime used was from 1,500 to 1,800 pounds per 1,000,000 gallons.

46. Precipitation Plant at Worcester.—In Worcester, Massachusetts, there is a chemical-precipitation plant with a capacity of about 15,000,000 to 20,000,000 gallons of sewage per day. An average of about 1,200 pounds of lime per 1,000,000 gallons is used as a precipitant, and about 50 to 55 per cent. of the putrescible organic matter in the sewage is removed. The sludge precipitated amounts to about 1¼ per cent. of the entire flow of the sewage.

47. Disposal of Sludge.—One effect of chemical precipitation is to form in the bottom of the tanks a large mass of semiliquid matter, temporarily arrested in the natural process of decomposition, containing about 90 per cent. of water and a large amount of alkaline precipitate. The proper and economical disposition of this sludge has been one of the serious difficulties connected with the chemical-precipitation process. A little more than 1 ton of pressed sludge (that is, sludge with one-half the water squeezed out) may be expected for each 1,000 persons per week; so that a city of 100,000 inhabitants would have to deal with about 125 tons per week, or 18 tons a day, of pressed sludge or double that amount of liquid. The most satisfactory method of disposal has been to run the sludge into boats and let the boats carry it to sea and dump it. When pressed, the sludge can be carted away to fill up low land, or to be mixed with certain kinds of farm land, though the manurial value of the sludge is very small. In still other cases, the liquid sludge is run on beds to a depth of about 1 foot, is allowed to dry out, and is then plowed under, and the process repeated. It usually costs more to dispose of the sludge than it does to treat the sewage.

BIOLOGICAL PURIFICATION OF SEWAGE

48. Reduction of Organic Matter.—The modern method of sewage purification, which may be called the **biological method,** rests on the principle that organic matter is mineralized by micro-organisms. All methods by which organic wastes are transformed into harmless products through natural agencies may be classed under this head.

The most exhaustive studies on this question of biological treatment have been made. At the Massachusetts experimental station, a tank 17 feet in diameter and 6 feet in depth was sunk in the ground, out of doors, and filled with 5 feet in depth of sand, of such quality as would make good mortar. This tank was so arranged that the effluent from it could be collected for examination, and every day, for 14 years, sewage has been applied to the surface of the sand in this tank. Daily examinations, both chemical and bacteriological, have been made of the sewage that has been applied and of the effluent after passing through the sand. The processes that have gone on within the sand have also been carefully noted by chemical and bacteriological experts. The conditions under which the experiments have been carried out have been practically the same as those found in any outdoor area in that climate, and it is fair to presume that the same results that have been obtained in these experiments can be obtained on a larger scale with any area of sand of like character. No part of the sand in the tank nor of the solid matter in the sewage that has been discharged into this tank has ever been removed. No vegetation has been allowed to grow on the sand. About 1,250,000 gallons of sewage, which is enough to fill the spaces in the sand about six hundred times, has been distributed on this limited area of sand during the 14 years it has been in operation. This amount of sewage has contained about 60 tons of sludge, which is enough to fill the spaces in the sand about ten times. The effluent from this sand filter during the experiments was purer than many drinking-water supplies, and the last published analyses, after the tank has been in operation 14 years, indicate that

the sewage that was applied to it in 1901 was freed from 89 per cent. of its organic impurities.

At first thought, this purification might be attributed to the fact that the sewage is strained through the sand. Such is not the case, however. Most of the organic impurities have been absolutely destroyed or transformed into other and inoffensive combinations, mainly through the action of bacteria. Obviously, this purification cannot be attributed to any process of straining, since, during the time the filter has been in operation, there has been no accumulation of solid matter within it, and the solid matter that has been applied to it would fill it many times, as has already been stated.

49. Filtration Through Broken Stone.—Another and still more interesting experiment, and one that completely disproves the theory of purification by straining, is the following: A filter was constructed of broken stone varying in size from ½ inch to 2 inches in diameter (too coarse to hold back the fine organic particles in the sewage), and sewage was passed continuously through this stone filter at a rate of about 2,000,000 gallons per acre daily. Only 15 per cent. of the organic impurities in the sewage appeared in the effluent.

After more than a year's continuous use, a casual observer on taking the crushed stone in his hand would note no change in it. A microscopic examination, however, would reveal a thin film on the surface of the particles. It is in this film that the bacteria live that cause the purification.

50. Vegetation and Sewage Purification.—The principle of applying sewage to the soil for the purpose of purifying it is not new. It is as old as agriculture. Formerly, vegetation was supposed to be an important and necessary factor in the purification effected. This supposition, however, is now disproved by the foregoing experiments. It has been shown that smaller areas of land will suffice where there is no vegetation, and a knowledge of this fact has led to the development of what is called **intermittent filtration**, which consists in applying sewage intermittently to porous and well-aerated soils in such a manner that air may be freely mingled with it.

51. Succession of Changes.—There are countless varieties of microscopic forms of life, each of which requires for its fullest activity certain conditions of food and environment, which it finds in some stage in the process of sewage purification, and there it establishes itself in its fullest vigor. The transformation of organic matter into harmless forms by these agencies is, therefore, a succession of processes, and occurs by a series of gradations regarding which there is still much to be learned.

It is apparent that an understanding of the habits of these various bacteria will enable us to supply the conditions that will render their aid most effective.

If a series of trays containing sand or some similar material are arranged vertically one over the other and sewage is allowed to trickle slowly down through the series, there will be developed in each of these trays colonies of bacteria suited to the conditions in which they find themselves, and the sewage as it flows from the last tray will be purified to a certain extent. Now, if the order in which these trays are placed is changed—that is to say, if the last tray is placed first in order—the bacteria will be subjected to different conditions; those that were receiving comparatively purified sewage will be receiving raw sewage, and corresponding changes of environment will occur in all the trays. Under these conditions, the bacteria become comparatively inactive, which results in a less degree of purification of the sewage. If, however, the trays are left in this changed order for a sufficient time, new colonies, better suited to the new environments, will establish themselves and become active, and the degree of purification will gradually increase until it reaches the same degree as formerly.

52. Fermentation.—Another step in the process of purification is that due to the action of anaerobic bacteria, or bacteria that thrive only in solutions of organic matter from which air is excluded. If sewage is held in a tank for a sufficient length of time, a process of fermentation will begin; certain changes will be produced that affect mainly the carbonaceous compounds in the organic matter, and a

considerable portion of the carbon, together with other substances, will be given off in the form of gas. There will also be a disintegration or liquefying of the solid organic matter of the sewage. The solid organic refuse, as the process of fermentation proceeds, will be entangled with the gases disengaged from it, and will rise to the surface, where the entangled gas will escape, and the solids, having lost their buoyancy, will again sink. This alternate floating and sinking will proceed until disintegration has proceeded so far that there is no longer organic matter sufficient to supply the buoyant gases, and the residue, which will be mainly mineral matter, will collect in the bottom of the tank.

If no fresh material is added to the tank, the bacteria that cause these transformations, having no further food supply, and having created by their vital action an environment in which they cannot thrive, will die and the process will be at an end. If the tank is supplied at a uniform rate with material of the same composition, a uniform condition will be established. If the composition of the supply or the rate at which it passes through the tank is varied, the bacterial life, and consequently the character of the effluent, will vary. These facts have been taken advantage of in the following manner:

Sewage is allowed to flow continuously through a tank at a rate that would change its contents from about once an hour to about once in 24 hours, according to circumstances. After a sufficient time, the anaerobic bacteria become thoroughly established in the tank. It has been determined experimentally, and also on a very large scale in actual practice, that, by passing sewage through such a tank, properly arranged, from 50 to 80 per cent. of its organic impurities may be removed, that it may be freed almost entirely from suspended matter, and that any suspended matter that may pass out of the tank will be so finely divided as not to be perceptible except so far as it gives turbidity to the effluent.

53. Oxidation.—If the effluent from a tank such as has just been considered is turned into a running stream having a comparatively rapid flow, so that its waters may be fairly

well aerated, or if the effluent is spread on the surface of the ground, or passed through porous filters, or in any other way exposed to the air, particle by particle, so that there are numberless points in contact with oxygen, still further transformations will take place, and the product will be comparatively stable and harmless.

54. Practical Application of Bacterial Purification.—All methods of biological sewage purification are founded on natural processes analogous to those just outlined. The experiments detailed have been selected as typical of processes used on a larger scale in works for the biological purification of sewage. As it is ordinarily discharged by a community, sewage varies in composition and quantity; and the organic life swarming in it is modified by these variations, so that in practice it is not feasible to obtain, by any of the processes just described, an effluent of unvarying composition, or one in which the bacterial activities that influence its further purification are unvarying. Aside from these local variations, there is also a marked difference in the sewage of different communities, and in the wastes from different processes of manufacture; and it is for the engineer to supply the conditions that will encourage bacterial activity along the lines and in the order of succession that will best serve his purpose. It is apparent that these conditions may be modified with advantage to suit particular cases, and that there is a certain time and sequence that will be most effective for each case.

BROAD IRRIGATION

GENERAL DESCRIPTION AND FEATURES OF THE SYSTEM

55. Land Treatment in General.—Since bacteria are to be found in all soils, and since by passing water through soils the water is finally divided and rapidly oxidized, it follows that land treatment in one form or another is the most natural way of applying the bacteriological processes

that have been outlined. There are at present three ways by which the sewage is applied to land; namely, *broad irrigation*, *intermittent filtration*, and *contact beds*. These three processes will be taken up in detail.

56. Broad Irrigation.—The term **broad irrigation** is applied to the spreading of sewage over a large surface of agricultural land, for the double purpose of purifying the sewage and utilizing the manurial properties of the sewage. Practically, it is also understood to mean the purification of sewage on clay soils, or on other similar soils so impervious that filtration is impossible. It means, therefore, the application of sewage to land in a thin sheet that will, in flowing over the surface, be so broken up that bacterial action may take place. Impervious soil and agricultural utilization of the sewage are therefore the characteristics of this process. The term is, however, also loosely used to denote the application of sewage to arid, sandy areas that by irrigation can be made to yield large crops. The sewage is really purified by filtration, but the agricultural results are so in evidence that the process is described as broad irrigation.

A high degree of purification may be accomplished in this manner, but the method is open to the following objections: (1) the difficulty of securing suitable land near centers of population; (2) local opposition and high prices generally demanded for the required land; (3) the failure, under these conditions, of making the cultivation of crops remunerative; (4) the difficulty of harmonizing the growing of crops with the demands of sewage purification at all seasons; (5) the expense of pumping sewage to remote and elevated areas.

57. Opposite Requirements of Sewage Disposal and Agriculture.—The purification of sewage by irrigation demands a constant and regular application of the sewage to some area. Growing crops forbid the application of sewage at certain periods of their growth, and at times when the natural rainfall alone oversaturates the soil. On account of these opposing requirements, it is necessary to have, as an adjunct to areas on which crops are cultivated,

either porous areas on which no crops are grown and to which the sewage may be applied when it cannot well be applied to fields under cultivation, or fallow areas to which the sewage may be applied with the purpose of enriching them for crops to be grown later. This method requires a very large extent of land, since it chiefly depends on the *surface* for purification. A greater area is required when the soil is clayey and retentive than when it is sandy and open.

58. Action of Bacteria.—The purification of sewage by broad irrigation depends mainly on aerobic bacteria, which, by their action, change the organic matter in the sewage into forms required by vegetation. The natural habitat of these organisms is the part of the soil that is exposed to the air, and in proportion as the air is excluded from the soil, their numbers and activity decrease. Therefore, porous soils are most effective in the purification of sewage.

A test made in one of the experimental filtration tanks of the Massachusetts State Board of Health, to determine the distribution of bacteria at different depths, gave the results shown in Table VIII.

TABLE VIII

NUMBER OF BACTERIA FOUND IN 1 GRAM OF SAND AT VARIOUS DEPTHS

Distance From Surface Inches	May 22, 1889	Distance From Surface Inches	May 22, 1889
0 to $\frac{1}{2}$	1,760,000	5	63,400
$\frac{1}{2}$ to $\frac{3}{4}$	105,000	8	30,700
$1\frac{1}{4}$ to $1\frac{1}{2}$	207,200	12	34,100
2	60,200	19	12,300
3	111,300	60	4,100

These determinations were made in soils much more porous than are usually available. Tests made in clayey soils indicate that below a depth of 12 to 18 inches there is little bacterial action.

It follows from the foregoing considerations that aeration of the soil is important, and, in order to maintain this, it is evident that the soil must be well drained, and that, if the best results are to be obtained, the surface must not become encrusted or coated with sewage sludge. In the most successful plants of this kind, it is customary to remove a portion of the solid matter from the sewage by sedimentation, by precipitation, or by rough straining of the sewage, so as to prevent the felting over of the surface. The sludge thus removed from the sewage is then spread on porous areas specially prepared to receive it, or is dug into well-drained areas, which may subsequently be cropped and cultivated.

59. Duty of Sewage.—The **duty of sewage** is the area of land the sewage will irrigate, which, of course, varies with the nature of the land. For instance, a loamy soil absorbs considerable water but retains most of it; hence, in irrigating such land, sewage or water can be applied only at comparatively long intervals. Sand, on the other hand, absorbs less water than loam, but gives up comparatively more, so that sandy soil can be flooded at more frequent intervals than can loamy ground, and, consequently, the amount of sewage that would irrigate a certain area of sandy soil would irrigate a much larger tract of loamy soil. In the case of loamy soil, a certain volume of sewage will irrigate a large tract, and the sewage under such condition is said to possess a **high duty,** while in the case of sandy soil where more sewage is required per unit area, the sewage is said to possess a **low duty.**

In utilizing sewage for irrigation, the duty is greatly dependent on the primary end to be attained. If the sewage of a large city is to be disposed of in the most convenient and inexpensive manner, a low duty is preferable; that is, the sewage is utilized on as small an acreage as will clarify it, the utilization of it for irrigation being simply incidental. On the other hand, there may be a great demand for the sewage because irrigation water is scarce, in which event as high a duty as possible is procured; that is, the sewage is applied to as large an area as it can be made to water.

60. Sewage is generally used to produce several crops in one season on the same soil. The theoretical amount that can be used in irrigating crops with benefit to agriculture is about 4.5 acre-feet to an acre (a depth of 4.5 feet on an acre). In the case of hay, of which several crops are grown in the same season on the same soil, as much as 12 to 15 acre-feet per acre may be applied without harmful effect. Even with such excessive application, it has been found that the average depth of deposit of solid matter in 10 years does not exceed ½ inch. In some cases, in the arid regions of the United States, sewage has been used in irrigation at the rate of 1 acre irrigated to 500 inhabitants. In other cases, the purposes being different, 1 acre has disposed of the sewage of only 150 inhabitants. Where sewage is most valuable for irrigation, it has been used at the low rate of 30 individuals per acre. Such duties as these illustrate the limits in amount of sewage that may be disposed of, rather than the ultimate duty of the sewage.

61. Fertilizing Value of Sewage.—House and city sewage is one of the richest of fertilizers. It must be applied within a few days, and the most practicable method is by dilution and carriage by water. By this means it may be moved 50 miles or more without deteriorating. The ratio of water may be as low as 10 cubic feet daily per inhabitant. Where sewage has been used in irrigation, crops are grown of such luxuriance that they increase the value of the land by from 100 to 400 per cent. Much of this beneficial effect, however, is due to the water rather than to the fertilizing effect of the sewage. Crops irrigated by sewage in the humid regions are found to be a little more luxuriant than those grown with chemical fertilizers. In the American arid regions, sewage produces better crops than water alone, but not much superior to those grown by the use of water supplemented by artificial fertilization.

62. Effect of Sewage Irrigation on Health.—The fears regarding the unwholesomeness of districts in which sewage is used for irrigation have been found to be largely

groundless where the sewage is properly handled. The combined action of vegetation and soil solves the problem of sewage disposal very satisfactorily. When the water charged with materials in suspension filters through the soil, the suspended matter is separated by mechanical action. The water containing nitrates reaches the roots of the plants and fertilizes them. Such water as is not absorbed descends to the subsoil and in the course of its passage is oxidized, the organic substances being changed into harmless nitrites and nitrates.

After much flowing of sewage, the soil must be thoroughly tilled in order to incorporate into it the insoluble matter caught on the surface. This process, in combination with the absorptive and clarifying effect on the water in passing through the soil, guarantees the salubrity of the surrounding region.

63. Crops Suited to Sewage Irrigation.—A great variety of products, including fruits, flowers, and vegetables, may profitably be grown on sewage-irrigated land. The most suitable crops, however, are cabbage, cauliflower, turnips, and hays. Alfalfa, or, as it is also called, Lucerne hay, may be cut as many as four or five times a season, and turnip crops have produced as high as 40 tons per acre, under sewage irrigation. Indian corn, sown for forage, is well suited for sewage farms, and it is supposed that in this way, and under favorable circumstances, from 30 to 60 tons per acre can be produced by growing two crops in a season.

Grasses and cereals are considered the most suitable crops for irrigation areas, owing to the fact that they are capable of absorbing and transpiring a larger quantity of sewage. Italian rye grass is said to absorb the largest volume of sewage, occupy the soil so as to choke down weeds, come early into the market, and bear frequent cutting. Leguminous plants are capable of taking up nitrogen from the air, and, as the object of growing crops on the area irrigated with sewage is to take up the nitrogen in the sewage, these crops are not suitable for this purpose. There are few places in the United States where sewage is systematically

used in irrigating growing crops. On a few such areas, however, ordinary field corn has been satisfactorily grown.

When the area under cultivation is large compared with the number of persons per acre that contribute sewage, all ordinary crops may be grown, since the sewage may be applied in moderate doses and to certain crops at the most advantageous time. But where the area is limited, and the sewage of more than 100 persons is deposited on an acre, crops must be grown that are specially adapted to receive large quantities of water.

Broad irrigation will be most successful where there are suitable areas of land—the more porous, the better—with a level or slightly inclined regular surface, where the rainfall is light, and the winters mild.

64. Effects of Temperature on Sewage Irrigation. It is natural to suppose that the processes of sewage purification would be greatly impeded, if not arrested entirely, by a very low temperature causing the ground to become frozen for a considerable depth. It nevertheless is true that they may be successfully carried on in climates where the average and minimum temperatures are very low. To account for this fact, it must be remembered that the sewage itself, as it flows from the sewer, has a relatively high temperature. At Lawrence, Massachusetts, it was found that when the mean January temperature of the air was about 15.50° F., that of the main sewer was 46.50° F. Continuous and heavy discharges of a liquid at this temperature would greatly tend to keep the ground open. In heavy snowfalls, the effluent sewage flows under the snow, and if ice is formed it will be in a thin sheet, under which the sewage runs, and is absorbed by the unfrozen earth underneath.

MANNER OF APPLYING SEWAGE

65. Sewage cannot be continuously disposed of on the same piece of land without harm to the crops. It must, therefore, be distributed with some system from one plot of ground to another so as to allow the soil to be cultivated and

crops grown. Sewage farms are therefore generally divided into many small tracts of one acre or two each, separated by distributing channels, so that the sewage may be delivered to one or more plots as conditions require. For the best effect, the subsoil should be comparatively porous, or else be well drained.

When the ground is nearly level, either the *ridge-and-furrow* or the *pipe-and-hydrant system* is that most usually adopted. On sloping ground, the *catch-work system* is most used. For intermittent filtration, when it is desired also to raise crops on the filter beds, the system of *absorption ditches* is employed. These systems will now be briefly described.

66. Ridge-and-Furrow System.—Fig. 1 shows a cross-section and plan of a portion of a field laid out on the ridge-and-furrow system: *M, M* are the main ditches, which may be any convenient distance apart, say 50 to 75 feet, according to the topography. The intervening space is laid out, as shown, in a series of alternate ridges and slopes, artificially produced by grading. Along the top of the ridges are placed the supply ditches *a, a, a*. The sewage flows into these from the main ditches, and, as they are kept nearly level, the liquid overflows in a thin sheet down the sides of the slopes, all that is not absorbed entering the drains *b, b, b*, through which it flows to the next main ditch below. The ends of the ridges are also sloped off, as shown in the plan, so that there is a flow from *a, a, a* in that direction also. The distance apart of the ditches *a, a, a* may be from 30 to 40 feet, and sometimes greater. The slopes of the ridges may be from $\frac{1}{10}$ to $\frac{1}{100}$, according to circumstances. The flow through these various ditches and drains is controlled by suitable gates. Although in the figure the ditches and drains *a, a, a, b, b, b* are shown as having a uniform width, they should be more or less tapering, the ditches *a, a, a* getting narrower and narrower and the drains *b, b, b* wider and wider as they run from one main ditch *M* to the next below it. In a large tract, the main ditches *M, M* will also be connected by other mains running between them, through

which the flow can be diverted as required, and many minor details will be found necessary beyond the general features shown in the figure. These details will be introduced here and there as needed after the system has been put in operation.

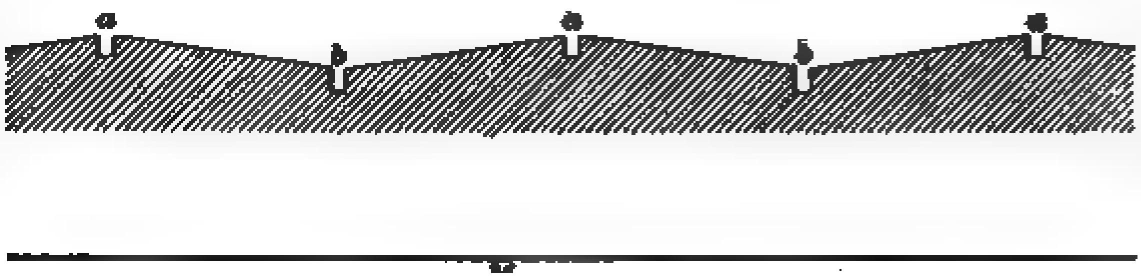


Fig. 1

67. Pipe-and-Hydrant System.—In the pipe-and-hydrant system, the sewage is conducted through a system of piping to all parts of the field, and suitable hydrant

outlets, to which hose can be connected, are provided at proper intervals. The pipe-and-hydrant system is one of the most expensive to install, and requires more help to operate than does any of the other methods of sewage irrigation commonly employed. For these reasons it is not extensively used.

68. Bed-and-Ridge System.—Land is sometimes laid out, as shown in Fig. 2, into beds of considerable extent,

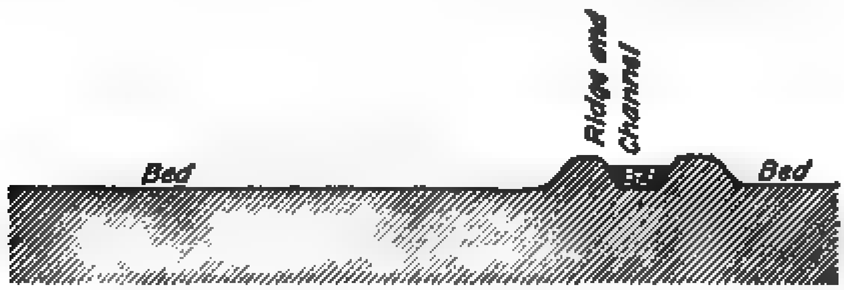


FIG. 2

alternating with ridges, along which the sewage is conducted in channels from which the beds are flooded. These beds can only be utilized for the growing of crops that will bear periodic flooding; consequently, this system is not so commonly adopted as the ridge-and-furrow system, in which sewage is applied without being allowed to cover the vegetation.

69. Catch-Work System.—The catch-work system, Fig. 3, is adapted to land having a sufficient slope to render

FIG. 3

the use of the ridge-and-furrow system impractical. In the catch-work system, a main ditch is dug following the line of highest level, and crossing the slope, therefore, approxi-

mately at right angles. The lower edge of this ditch is kept nearly level, so as to permit the liquid flowing through it to overflow in a thin and even sheet. This overflow is produced at the points where it is wanted by placing a temporary dam or partial obstruction of some kind in the main ditch. At a certain distance below the main ditch, another smaller one is prepared, also following a level contour line. This ditch catches the unabsorbed overflow of the main ditch, and in turn is made to overflow in the same way, irrigating a lower belt of the slope. A succession of these smaller ditches convey the sewage progressively from the top to the bottom of the slope or hillside. It is evident that by these successive checks the liquid material is prevented from acquiring a dangerous velocity, which would wash away the earth and create a good deal of damage.

This method seems to be considerably cheaper than any of the other three previously described, and, consequently, is to be preferred when the slope is sufficient, the ridge-and-furrow system being used only when the land is too flat to admit of the catch-work system.

70. Absorption-Ditch System.—The absorption-ditch system resembles greatly the simple furrow system. It may be used to advantage when it is desired to cultivate crops on intermittent filter beds, and presupposes a regular and nearly level surface. In this system, the filter bed is laid out in a series of parallel ditches of varying dimensions and distances apart. Ditches 12 inches wide and 5 feet apart from center to center have been used, though the dimensions may vary greatly. Sewage is admitted into these ditches from the supply conduit, and slowly permeates the soil, the purified effluent passing off through the drain pipes with which the filter bed is underlaid, and the fertilizing substances remaining in the ground. The intervening strips between the ditches, which are fertilized by lateral absorption, can be cultivated to advantage. Corn would seem to be a very suitable crop when filter beds are laid out in this way.

The absorption-ditch system, while it facilitates the cultivation of the filter bed, naturally requires a greater area than if the whole surface of the bed were flooded with sewage. Naturally, too, the ditches will frequently be gorged with more sewage than they can readily absorb, so that the danger of overirrigation at inopportune times will still exist, though to a modified extent. In order to avoid possible overflow, the filter beds are surrounded with low embankments.

INTERMITTENT FILTRATION

71. Intermittent filtration consists in applying large quantities of sewage at certain intervals of time to specially prepared and very porous areas of land, and allowing an interval for rest and aeration between applications. It is an outgrowth of the necessity of increasing the quantity of sewage that may be applied to a given area, and of the acquired knowledge that growing crops are not necessary to the purification of sewage. Although vegetation may have an important part in sewage purification, it materially interferes with the continued application of the sewage, and it has been shown that nitrates, which are an essential plant food and are formed from the organic nitrogen in the sewage by processes within the soil, are not necessarily harmful in a sewage effluent, and that their utilization in plant life is immaterial so far as the purification of the sewage is concerned. For this reason, and for the further reason that sewage farms ordinarily are conducted at a loss, intermittent filtration has come into general use. It is essentially a biological process.

72. Principle of Intermittent Filtration.—In intermittent filtration, purification is effected by microscopic organisms that are contained in the sewage or in the soil, and become established and active in the filter. Their activity is favored by porous soils, which readily admit air during intervals of rest. Filtration, as commonly understood, is really a misnomer for this process, since the purification is

due only in a very limited sense to straining. There is a continued and absolute destruction of the organic matter, as such, in sewage by these organisms. Otherwise, the filters would soon be obstructed with the accumulation and lose their effectiveness. (See *Purification of Water*, Part 1.)

73. Material for Filter Beds.—Suitable material for intermittent filtration is sometimes found in place; but generally the areas to which sewage is to be applied are specially prepared for the purpose. The filter beds consist of a layer of sand, or sand and gravel about 5 feet deep, surrounded by embankments to confine the sewage within the limited areas when the beds are flooded. A number of beds are usually provided, and they are flooded in a certain rotation, leaving a considerable interval for the sewage to pass down through the sand and for the sand to become aerated after the sewage has seeped away. The surface of the beds is generally made perfectly level; but where they are formed on sloping ground, there is generally a succession of tiers at different levels. Sometimes, where suitable surface soil for the purpose cannot be found (for instance, where there is a compact soil underlaid by a sandy or a gravelly subsoil), the surface earth is removed and formed into embankments to separate the beds, and the underlying sand exposed.

74. Absorptive Power of the Soil.—From experiments by Doctor Lissauer to determine the absorptive power of soils, the following conclusions were derived:

"1. The liquid entering the pores of the soil displaces air or liquid previously present, forcing the former upwards into the atmosphere, and the latter downwards into the subsoil or effluent water.

"2. In order that the effluent water may not be directly polluted by the sewage liquid, the maximum supply of the latter must not be more than can be taken up by the pores of the soil.

"3. Dry loamy soil absorbs more than peaty soil and gives up less; while dry sandy soil, on the contrary, absorbs less and gives up more. Consequently, a loamy soil, though it absorbs

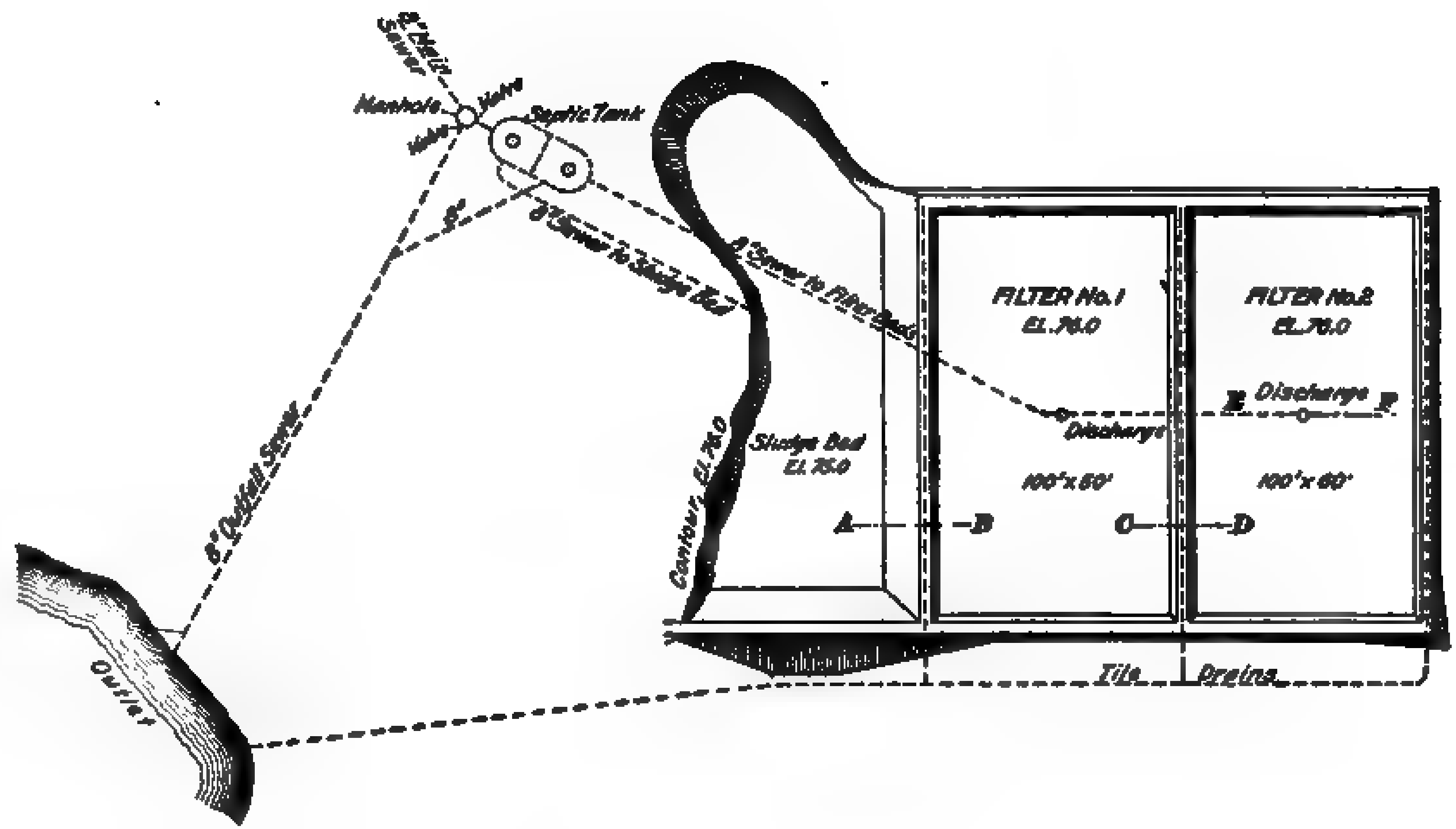
a large quantity of liquid, can seldom be irrigated; whereas a sandy soil, though it absorbs but little, may often be irrigated.

"4. The looser the soil, the more easily are watercourses formed in it, and therefore the less can its maximum power of absorption be approached; otherwise, the sewage liquid might penetrate the subsoil before the whole of the ground had been saturated.

"5. In order, therefore, that the effluent water may be protected from pollution, it is especially necessary that the absorptive power of the soil should be known; but the determination is of no value unless it is made in a sample in which the natural position of the particles of earth has been undisturbed."

75. Rate of Application: Sludge.—The rate of application of sewage to a filter is influenced materially by the amount of sludge contained in the sewage. High rates cannot be maintained continuously if sludge is allowed to collect on the surface of the filter. If it is allowed to collect in this way, frequent intervals of rest must be afforded, so that the sludge will dry and become porous, or it must be dug into the surface of the filter so as not to prevent the access of the sewage or the entrance of air into the interstices of the filter. Where high rates are demanded, it is customary to remove a portion of the sludge by sedimentation, precipitation, or filtering through some coarse material, and thus the process of intermittent filtration becomes a secondary step. It is very commonly used in this way and with great advantage.

A rate of from 50,000 to 100,000 gallons per acre per day is probably as high as can be maintained with ordinary sands when the sewage has been freed from no part of the sludge; and, in order to maintain this rate, the surface of the filters must be frequently raked over and the solid matters occasionally dug into the soil. Rates as high as 500,000 gallons per acre per day have been maintained for considerable periods where the sludge has been previously removed from the sewage.



A—B Sections C—D E—F

Fig. 4

76. Method of Applying the Sewage.—In large-sized plants, where the expense of constant attendance is justified, the sewage is generally admitted to the beds in rotation by gates operated by hand. If the beds are large, carriers are arranged so as to distribute the sewage uniformly over the entire surface. In small plants, where constant attendance is not practicable, it is better to discharge the sewage automatically, which can be easily done by automatic flush tanks placed in a collecting chamber.

An intermittent-filter plant is shown in Fig. 4, which is substantially taken from "Engineering News." Sewage is delivered to the works through an 8-inch main sewer that is provided with two valves located in a valve chamber on the site of the purification works. One valve controls an 8-inch outfall sewer, through which sewage can be discharged, without purification, into the stream; while the other valve controls the flow of sewage to the septic tank and filter beds. (Septic tanks will be described further on.) From the septic tank, a line of 8-inch sewer pipe is extended to the two filter beds, where it is provided with one opening located in the center of each bed. In addition to the two filter beds, the purification plant is provided with a sludge bed, into which the accumulated sludge from the septic tank is flooded at intervals through an 8-inch sludge pipe. From the discharge compartment of the septic tank, an 8-inch pipe is extended to and connected with the 8-inch outfall sewer, so that, when necessary, unfiltered septic effluent can be discharged direct into the stream. The effluent from the filter beds is collected by a system of tile drains that discharge separately into the stream. In addition to the plan of the sewage-purification works, the illustration shows a section through parts of the filter and sludge beds, drawn to a larger scale than the plan.

77. Intermittent Filtration in Winter.—Cold weather does not interfere with the process of intermittent filtration so seriously as it does with broad irrigation. This is due to the fact that much larger quantities can be

successfully applied to the same area, and the temperature and greater quantity of the sewage prevent the formation of ice. A slightly smaller percentage of organic matter will be removed, however, in winter than in summer. In northern latitudes, the surface of intermittent filters is often ridged, so that, when a coating of ice forms over it, the ice rests on the surface of the ridges, leaving abundant waterways underneath through which the sewage is distributed. Much depends on the temperature of the sewage as it is delivered at the filters, and this will, of course, be affected by the remoteness of the filtration area, and also by the quantity of sewage that is delivered through the main sewer. Fine soils are not adapted to intermittent disposal in winter.

SEDIMENTATION AND SEPTIC ACTION

78. Clarification of Sewage by Sedimentation. When sewage is allowed to stand quietly or to flow continuously into and out of a tank at a rate that will not agitate it, there is a marked purification due to sedimentation. Furthermore, the suspended impurities thus deposited gradually decrease in volume and lose a considerable part of their organic matter. This principle has been made use of in purification plants for the purpose of protecting filters, lessening the work they have to perform, and increasing their efficiency. It is only within a few years, however, that the principles on which this action is based have been at all understood and utilized to the extent they are at present.

79. Septic Tank.—Since the earliest attempts at sewage purification, in which broad areas of land were considered necessary, there has been a constant effort to devise means by which the work could be accomplished within a limited space, and this has led to the development of what is called the **septic tank** as one stage in the process. A septic tank, which is simply a tank where the sewage is allowed to flow and deposit some of its impurities by sedimentation, is generally used in combination with some process of rapid

filtration or oxidation, which naturally follows it, although in some instances, where no great degree of purification is required, the septic tank alone may be sufficient.

80. Advantages of Septic Treatment of Sewage. The advantages of the septic method of sewage treatment are:

1. There is in the tank a material reduction of the solids that otherwise it might be necessary to handle as sludge. This reduction has been variously estimated at from 40 to 50 per cent. of the total solids, and at from 60 to 80 per cent. of the organic portion of the solids.

2. Sedimentation materially increases the rate at which sewage may be filtered, and consequently decreases the area that must be devoted to filters.

3. The sewage undergoes chemical changes that peculiarly fit it for rapid oxidation in filters, and thus still further reduce the required area of filter beds.

4. Sewage can flow into and out of the tank at the same level, and thus no loss of grade is necessary. This is often an important consideration.

5. No attendance is necessary.

6. The sludge may be drawn off or pumped out, and is much more easily handled than when separated by any process of straining or by coarse filters.

7. Septic tanks have a material influence in equalizing the composition of sewage flowing through them. It has been observed that, while the composition of the sewage discharged into a septic tank varies widely at different hours of the day, the proportional amount of chlorine (parts per 100,000) in the effluent from the tank is almost constant; and it may be presumed that the proportions of other constituents of the sewage are equalized in much the same manner, and that, therefore, the sewage is better prepared for subsequent treatment in oxidizing filters.

81. Changes in a Septic Tank.—The action within a septic tank is similar to the decomposition of coal in gas retorts from which air is excluded; the chief products

are identical with the principal constituents of illuminating gas, and will burn with a blue flame. The reduction in the amount of organic matter is due mainly to the gaseous products that are given off in this way. The amount of gas given off has been determined in several instances: in experiments by the Massachusetts State Board of Health, it was found to vary from 4.6 to 8.4 per cent. of the volume of the sewage. Professor Kinnicutt found that at Worcester, Massachusetts, in an experimental tank, it varied from .09 to 7.7 per cent. of the volume of the sewage.

In a septic tank there is a continual agitation of the sludge, due to its entanglement with the gases that are disengaged from it; these, in rising, carry the particles to the surface, when the gases are disengaged and the solid matter again sinks. This process continues until the organic matter is reduced, the evolution of gas abates, and the solid matter, which is now mainly mineral, sinks to the bottom, where it remains. Occasionally, a thick scum forms on the surface of the liquid in the tank, sometimes to a depth of 6 or 8 inches. The formation of this scum was formerly supposed to be a necessary part of the process, but later investigations indicate that it may form and disappear without having any material influence on the degree of purification.

82. Closed and Open Septic Tanks.—It was until recently supposed that the septic process required tightly closed tanks that would exclude the air; covers, however, have been found unnecessary, as the action in open and closed tanks is practically the same, except that covering may maintain a more uniform temperature. The reason why an open tank is practically as effective as a closed one is that quiet deep bodies of liquid do not readily absorb air; hence, the liquid in an open tank is affected by air only at or near the surface.

83. Organic and Mineral Matter in Septic Tanks. Determinations of the proportions of ash and organic matter at various depths in a septic tank at Champaign, Illinois, gave the results shown in Table IX.

From this analysis, it will be seen that the solid matter that finally accumulates in the bottom of the tank contains but a relatively small proportion of the organic refuse. This feature may be taken advantage of by constructing septic tanks in such a manner that the lower stratum of sludge, which is largely mineral matter, may be drawn off without interrupting the operation of the plant, leaving the higher strata for further dissolution of the organic matter. Great care must be taken in this, however, since any general disturbance in the tank interferes seriously with the bacterial activity and decreases the value of the tank action possibly for weeks. In some instances, septic tanks have been in continuous operation for 4 or 5 years without removal of any portion of the accumulated sludge.

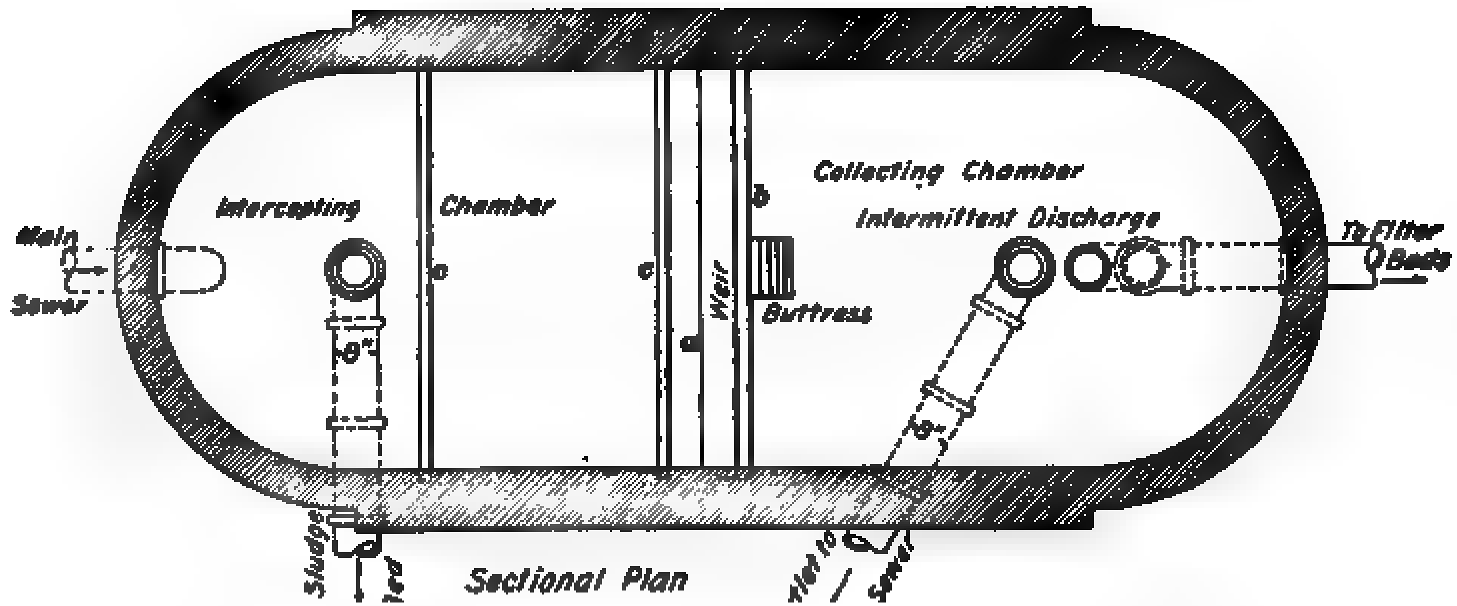
TABLE IX

ORGANIC AND MINERAL MATTER IN SEPTIC TANKS

Source of Sample	Organic Matter	Mineral Matter
Sludge from the bottom .	.12	.88
Half depth	.37	.63
Scum	.61	.39

84. Capacity of Septic Tanks.—Septic tanks are generally operated under a continuous flow. The size that should be provided depends on the character of the sewage, the distance of the tank from the center of population, the extent to which septic action has already progressed in the main sewer before the sewage reaches the tank, and the manner in which the effluent is to be further treated. Ordinarily, the tanks have a capacity sufficient to hold from 6 to 18 hours' flow of sewage, but they have been installed with capacities varying from 1 to 24 hours' flow. An 8-hour flow, or one-half the daily flow, is an average capacity for a septic tank.

After an effluent passes from a septic tank, and becomes exposed to the atmosphere, another class of bacteria—those that promote oxidation—become active, and it is important



Sectional Plan

Transverse Section.

Fig. 6

that the process within the tank should not be carried so far as to entirely destroy this class of bacteria or to reduce their vitality to such a degree as to render them useless in subsequent processes.

85. Constructive Details of Septic Tanks.—Septic and sedimentation tanks should be arranged so that the inflow and outflow of sewage will create but little agitation. Both the inlet and the outlet should be below the water-line, and the effluent should be drawn off from the level at which the liquid is most free from suspended particles. Sedimentation is more active where there is a considerable depth of sewage.

86. Upward Filtration Tanks.—Tanks have sometimes been filled with coarse material and operated as anaerobic filters, generally by upward filtration. This offers a greater area for the lodgment of bacteria, but greatly decreases the sewage content of the tank and makes it very difficult to remove the sludge. Sewage was purified experimentally in this way by Frankland as early as 1870, and in his experiment about 44 per cent. of the organic matter was removed. This process, however, is expensive, and unsatisfactory in other respects.

87. Example of a Septic Tank.—In Fig. 5, taken from Engineering News, are shown a plan view, longitudinal section, and transverse section of a septic tank. The plant consists of one large covered masonry tank divided into two compartments by a wall *a* that separates it into an **intercepting chamber,** which is the septic part of the tank, and a **collecting chamber,** where the effluent is collected ready for intermittent discharge to filter beds, contact filters, or other place of disposal. An automatic siphon is usually provided for discharging the effluent from the collecting chamber. When the effluent reaches a certain height, it sets the siphon in operation, which continues until the fluid is exhausted from the chamber. The dividing wall forms a weir for the septic effluent, and, to prevent wear, the top is furnished with a metal rail *b* bedded in cement concrete. Baffle boards *c*, *c* are provided to keep the surface scum from

overflowing the weir into the collecting chamber. The inlet to this tank is above the sewage level, but dips down on the inside to convey the inflowing sewage below the surface scum. The bottom of each compartment slopes toward the center, where the inlet to a pipe is located. In the intercepting chamber, the inlet is to a sludge pipe that is provided in order that the sludge may be washed out without throwing the tank out of service. In the collecting chamber, the inlet is to a by-pass, through which septic effluent can be discharged into a stream or other place of disposal without passing it through the filters.

OXIDATION IN POROUS MATERIAL AND AT HIGH RATES

88. Action of Coarse Filters.—Oxidation is the final process of sewage purification. By providing filters of very porous material—such as coke, cinders, broken stone, or crushed slag—so that air is freely admitted into the interstices of the filter, it is possible to oxidize the organic matter at a very rapid rate; and, if the sewage has been previously treated in septic tanks or anaerobic filters, or by some process that breaks down the particles of organic matter, so that they may be in intimate contact with the air and with oxidizing bacteria, particle by particle, the rate may be still further increased. The case is similar to the combustion of wood. A whole log cannot be rapidly burned, because only its surface can be exposed to the flame; if it is finely divided, it exposes a much larger surface of contact and will be consumed in much less time; and, if it is reduced to the condition of floating dust, it may be consumed in a single flash.

The object of oxidizing filters that are to be operated at a high rate is to expose the greatest possible surface of liquid to bacterial action, and, at the same time, to contact with air. Contact with air is necessary in this process, because oxygen is indispensable to the life and activity of the oxidizing bacteria.

89. Material for Coarse Filters.—The material most commonly used in **bacteria beds**, as these coarse filters are

more properly called, are coke, cinders, crushed stone, gravel, crushed slag, burned clay, and broken brick. The material should be hard and indestructible, not affected by frost, and not soluble. If there is any breaking down of the material, the pores of the filter become clogged, and its effectiveness, particularly at high rates, is greatly reduced.

There is an advantage in material consisting of hard angular fragments over material consisting of round and regular fragments forming a smooth surface. This is illustrated by Table X, which shows the results obtained by the Massachusetts State Board of Health.

TABLE X

ANALYSIS OF EFFLUENT FROM COARSE FILTERS

(Parts per 100,000)

Material of Filter	Albuminoid Ammonia	Oxygen Consumed	Nitrates	Bacteria per Cubic Centimeter
Coke	.091	.48	1.13	80,900
Glass beads . .	.169	1.07	.51	191,900

"The better purification obtained by the coke filter was due undoubtedly to the rough surface of the coke, by means of which it not only holds more air than the glass beads, but also retards the flow of sewage, so that it is a greater time passing over the coke than over the beads, and the matters in suspension are undoubtedly held back more effectively than by the glass beads."

These filters were operated at an average rate of 738,400 gallons per acre per day, and under precisely similar conditions except as to the material of which they were composed.

The material that should be used in each case will depend on the accessibility of various materials. It often may be advisable to use a material, on account of its less cost, that is not the most effective. For instance, gravel is much less effective than coke or cinders, but gravel is often found in place, and beds of this material can then be prepared with

but little grading, and what the gravel lacks in efficiency can easily be made up by increased areas.

90. Distribution of Sewage on Coarse Filters. Theoretically, the proper way to distribute sewage on a filter bed is to apply it continually and regularly at every point on the surface of the bed, in such quantities that the aeration of the interstices of the bed shall be sufficient, and the conditions within the bed shall always be constant. This cannot well be accomplished in practice, however, on account of the variable flow, and because it would require expensive appliances to do it on a large scale. It is sometimes done in small plants by means of revolving sprinklers operating in circular beds, or by traveling distributors passing alternately back and forth over rectangular beds.

Sewage is often distributed on coarse filter beds through a system of pipes that branch out to various points on the bed and are left with an open end through which the sewage is discharged. In coarse material, such as these beds are made of, the sewage does not spread out laterally so as to utilize all the filtering material, but tends to pass rapidly downwards and out through the drains. When sewage is distributed by this means, it is generally applied intermittently from two to four times a day, and the bed is occasionally allowed a whole day for rest and more complete aeration.

91. Contact Beds.—The difficulty of bringing sewage into intimate contact with all the filtering material in coarse beds is obviated by what are known as **contact beds.** These are built in the same way as other coarse filters, except that the outlet of the underdrain is provided with a gate that is closed while the bed is being filled, and is kept closed for a certain period—generally about 2 hours—after the bed is filled. The gate is then opened, and the bed drained and allowed to aerate until the next filling. By this means, air is drawn into the body of the filter in sufficient quantity to oxidize the sewage. Contact beds can usually be filled from two to four times each day.

92. Capacity of Contact Beds.—Contact beds depend for their capacity on the volume of interstices of the material of which they are composed. This volume varies from 25 to 45 per cent. when the beds are first made, but decreases invariably as sewage is filtered. This loss of capacity is the most serious difficulty experienced in managing contact beds, and is due to the gradual breaking down or settlement of the material, to the accretion of bacterial slime on the particles, and to the accumulation, in the voids, of fine silt continually carried along by the sewage. The first and last factors are cumulative, and result in so decreasing the efficiency of the beds that they must, in the course of a few months, be entirely renewed, unless preventive. measures are adopted. Those measures are: (1) the special selection of vitreous flinty material, which will break down but little; and (2) the removal of all the silt and detritus before the sewage goes to the contact beds. Even with these preventives, there is a certain loss of capacity on account of the accumulation of organic matter, but this loss never goes above a certain amount in well-managed plants, since the bacterial action destroys the organic matter as fast as it accumulates.

Some conception of the volume that can be purified by contact beds may be had from the following figures: An acre of sand 3 feet deep contains 130,680 cubic feet, and the voids, at 40 per cent., represent a volume of 52,272 cubic feet, or 392,000 gallons. If the beds are filled twice a day, the capacity of the bed will be 784,000 gallons per acre per day, the beds having their full volume of voids. This volume will, however, be decreased about one-half, so that the net capacity of such beds, under the best conditions of material and previous treatment, is about 400,000 gallons per acre per day.

93. London Experiments.—In the Fourth Report on the Experimental Treatment of London Crude Sewage, 1902, the following conclusions are stated:

1. That, by suitable, continuous, undisturbed sedimentation, the raw sewage is deprived of matter that would choke

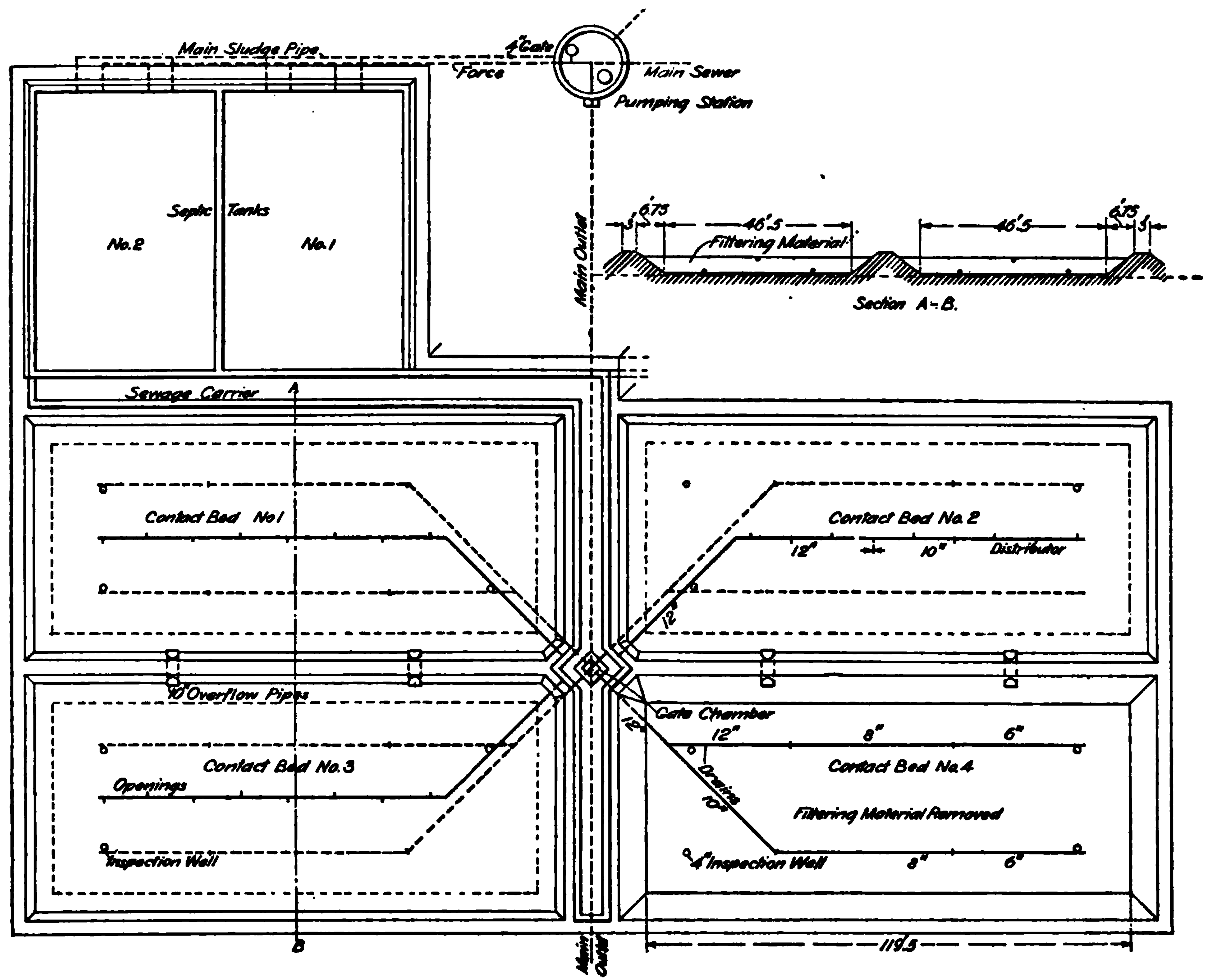

Main Sludge Pipe
4" Gate
Force
Main Sewer
Pumping Station
Septic Tanks
No. 2
No. 1
Main Outlet
6.75
3
46.5
Filtering Material
46.5
6.75
5
Section A-B.
Sewage Carrier
A
Contact Bed No 1
Contact Bed No. 2
12"
10"
Distributor
12"
10" Overflow Pipes
Gate Chamber
12"
12"
8"
6"
Contact Bed No. 3
Openings
Inspection Well
Contact Bed No. 4
Drains
10"
Filtering Material Removed
4" Inspection Well
8"
6"
B
Main Outlet
119.5

FIG. 6

the coke beds, and the sludge that settles out is reduced to a very considerable extent by bacterial action.

2. That the coke beds, after they have developed their full purifying power by use, have a capacity of about 30 per cent. of the whole space that has been filled by coke.

3. That the sewage capacity of the coke beds, when fed with settled sewage, fluctuates slightly, but undergoes no permanent reduction. The bed does not choke, and its purifying power undergoes steady improvement for some time.

4. That the bacterial effluent of settled sewage from the coke beds does not undergo offensive putrefaction at all, even in summer heat, and can never become offensive. That this effluent satisfactorily supports the respiration of fish.

5. That the use of chemicals is quite unnecessary under any circumstances when this method of treatment is adopted.

94. Arrangement of Contact Beds.—The general arrangement of a purification plant, in which the sewage is first treated in septic tanks and then passed through bacterial contact beds of coal cinders, is shown in Fig. 6 (from Engineering News). The sewage treated at the purification plant is handled by pumps located in a pump house situated on the premises near the septic tanks. The pumps can force the sewage into the septic tanks, or, when occasion requires, they can pump directly into the main outlet and thus by-pass the raw sewage without any purification to the main outfall. From the septic tank, the effluent is conducted by a sewage carrier to a gate chamber situated at the junction of the four contact beds. Gates are here provided to control the flow of septic effluent to the beds and the filtered effluent from the beds. One or all of the beds can be thrown out of service, and, when necessary, the septic effluent from the tank can be discharged at the sewer outfall, without previously passing it through the filter or contact beds. When in service, the beds are flooded in rotation, some time being allowed for filling; they are allowed to stand full during a period of about 2 hours, and then to remain empty during an equal period, for aeration. Sewage is distributed on the surface of

the beds through earthenware pipes laid on the surface of the cinders. The pipes are shown by dotted lines on beds *1*, *2*, and *3*; the underdrains are shown or indicated by solid lines. They are more clearly seen in bed *4*, from which the filtering material is shown removed to expose the pipes.

95. Example of Multiple-Contact Beds.—Fig. 7 illustrates a contact plant having separate contact beds and an automatic discharging device for controlling the flow of sewage and directing it alternately into the beds. In this plant there are four beds, of which three, marked *a*, *b*, and *c*, respectively, are shown. The fourth bed and the partition wall between it and the bed *c* have been omitted in the illustration in order to show the construction of the beds and the automatic controlling devices. Each bed is made of brick, and is cemented water-tight; the walls are laid about 30 inches high, and the area of each bed is large enough to contain, after being filled with broken stone to a depth of from 20 to 24 inches, the sewage of about 6 hours' flow. The sewage flows by gravity from the septic tank *d* through the inlet sluice *e* over the weir *f* inside the bell *g*, and then over the weir *h* into the inlet locking chamber *i*, compressing the air in the air bell *j* shown by dotted lines. As the sewage rises in the locking chamber, the air pressure in *j* is communicated through the pipe *k* to an inverted bell over the outlet-gate weir *l*. The air pressure depresses the water in the bell, thereby lowering the water level below the outlet-gate weir *l* and thus preventing the flow of sewage from the bed. When the chamber *i* is full, the sewage overflows into the contact bed *c*.

When the contact bed is full, the sewage overflows into the locking chamber *m*, displacing the air in the air bell *n*. This air passes through the air pipe *o* to the top of the bell of the inlet sluice *g*, closing it. At the same time, air in the small air bell *p* is displaced, and, passing through the air pipe *q*, releases the closed sluice gate in the filter bed *b*, unlocking the inlet and allowing *b* to take sewage from *e*. The sewage remains in the filter bed *c* until the timing chamber *r* has

IG. 7

8 93 20139

been filled through the timing cock *s*, shown dotted, which is set to give the desired time of contact. When the air in the small bell *t* is sufficiently compressed, it unlocks the seal of the outlet sluice, and the effluent is discharged either to the river through the sewer pipe *u*, or to secondary contact beds in case a higher degree of purification is desired than can be obtained in the primary beds shown. The outlet sluice of *c* remains open until the other three contact beds have been filled, when the small bell in the inlet locking chamber of the last bed filled blows the seal of the release trap *v*, thus relieving the air pressure and again admitting sewage from *e* to *c*. The air supply to the bell *n* is maintained by uncovering the bottom edge of the bell, the chamber *m* being siphoned empty through the siphon *w* every time the liquid is drained from the contact bed. The compressed air in the bell *g* cannot escape through *n* when *m* is empty, because the pipe *o* dips into the water on the sluice side of the plate *f*. The pipe *x* leading from the release trap *v* connects to the top of the bell *g*. The pipe *y* connects the inlet end of the release trap to the bell in the inlet locking chamber of the bed that is not shown.

In the illustration, the bed not shown may be assumed to be full; the bed *c* receives the flow of sewage and is filled nearly to the locking level; the bed *b* has been resting and aerating, and is ready to receive its next charge; the bed *a* is also resting empty, the sewage not removed at the first flow draining slowly through the outlet sluice that remains open until the next filling.

96. Straining Sewage Through Coke.—Sewage is sometimes strained through coke for the purpose of removing the suspended matter so that the effluent can be more readily handled in filters. Fine coke (coke breeze) is generally used for this purpose, and the strainers are from 6 to 15 inches thick. There is comparatively little bacterial action in this process. It does not materially reduce the amount of solids in the sewage, but merely retains them on the surface of the coke or in the upper layer, which is removed from time to

time as the strainer becomes clogged, and the coke and the solids that are mingled with it are burned. It has been estimated that, for each million gallons of sewage strained through the coke, about 150 bushels of coke must be replaced, and that from 30 to 50 per cent. of the organic matter in the sewage can be removed by this process.

Strainers can best be used where there are pumping stations in connection with the disposal plant, or where there are other conveniences for utilizing the heat obtained by burning the rejected coke.

97. Sprinkling Filters.—The necessity, in order to thoroughly oxidize the organic matter in sewage, of finely dividing and breaking up the volume of sewage as it goes through a filter, has led gradually to the custom of sprinkling the sewage on the beds, which then act as filters. This process, called in England the **percolating-filter treatment,** deals with sewage at high rates and requires a high degree of technical skill for its successful management.

The process varies in its details, although it always requires previous septic treatment. In some plants, small pipes are laid over the surface of the bed, through which the sewage is forced, under pressure, to escape through openings on top. On emerging, it impinges against a spatter plate and so falls back on the bed in fine drops or a spray. In other plants, a series of revolving sprinklers, similar to the mechanical contrivance known as Barker's Mill, or to a lawn sprinkler, are operated by the sewage under pressure. It was thought at first that the sprinkling process would obviate the necessity of resting the bed, but it is found practically that, while the period of rest need not be long, some interruption of the flow is necessary. The perfected type of apparatus consists, therefore, in a narrow platform or bridge spanning the rectangular beds. This bridge moves like a traveling crane, from one end of the bed to the other and then back, the reversal at each end being automatic. The sewage is discharged in a spray from the traveling platform, which traverses the length of the bed in a minute or so. In this

way, the sewage is discharged continually, while the different parts of the bed receive the dose intermittently. On proper beds, sewage may be dealt with at the rate of 2,000,000 gallons per acre for 24 hours.

98. Subsurface Irrigation.—It is often desirable, especially in the case of plants built for private residences, to conceal the area on which the sewage is discharged. This may be done by treating the sewage in a septic tank, and then discharging it into a series of agricultural tile drains laid about 10 inches below the surface. These drains are laid in parallel lines about 6 feet apart, leading out from a main at right angles to their direction. The drains should have a grade of from 1 inch in 50 feet to 1 inch in 200 feet, the steeper grades being used on the more porous soils. The sewage should be discharged through these drains intermittently, so that the soil may imbibe air between the applications of sewage. For this reason, and because the solid organic matter should be broken up and liquefied before it comes in contact with the soil, this, like all land processes, requires that two tanks be introduced between the house and the soil. The first tank should be of a capacity about equal to a day's flow of sewage, roofed, and arranged like the septic tank described in Art. 87. This tank remains continually full, and must remain so in order not to disturb bacterial processes. The second tank should hold the flow of about 4 hours at the time of maximum flow, and should empty automatically when full. The contents are thus discharged intermittently into the drains three or four times in 24 hours, which allows the ground to recover, between doses, its normal texture. The amount of land needed for this method depends on the character of the soil. In coarse sandy soil, with good loam on top, about 20 gallons per square yard per day should be provided, or 4 square yards per person. As the soil becomes finer and more alluvial, the quantity increases, until 1 gallon per square yard per day, or 80 square yards per person, is needed. Between these limits, only judgment and experience can determine the necessary

area. The direction of the parallel lines of tile is selected with reference to the contours, so that the desired grade may be obtained and yet the tile will be always the same distance below the surface.

PURIFICATION, SEPARATION, AND SLUDGE

99. If sewage is passed through sand or broken stone, or is held in tanks for the purpose of purifying it, the relative proportions of organic matter in the sewage and effluent may be fairly stated as in Table XI.

TABLE XI

ORGANIC MATTER IN SEWAGE AND EFFLUENT

Purifying Agent	Parts in 100,000		Per Cent. Removed
	In Sewage	In Effluent	
Sand, 50,000 gallons per acre per day	20	1.5	93
Broken stone, 1,750,000 gallons per acre per day	20	4.0	80
Tank	20	8.0	60

It is important to know what has become of the organic matter that does not appear in the effluent. If it has been merely *separated* from the sewage and accumulates in an offensive form, undiminished in quantity, the process of purification is not complete, and there still remains the problem of disposing of the sludge. It is therefore necessary, in comparing the results of different processes of purification, to consider not only the conditions of the effluent, but also the character and quantity of sludge remaining, and the means by which it can be disposed of. Less attention has been given to the character and composition of sludge than to sewage effluents, and the quantity that may be expected, and its character cannot, especially in biological processes, be closely estimated.

100. When sewage is applied to wide areas of land under cultivation, there is often no separation of sludge, the sludge being mingled with the soil. When sewage is applied to limited areas, as in intermittent filtration, and particularly where a high rate of purification is attempted, the sludge interferes with the rapidity of the process, by coating over the surface and excluding the air, and in this process the sewage is often allowed to settle and part with some of its suspended solids before it is applied to the filters. In the modern biological processes, where high rates are used, the suspended sludge is almost always previously separated, though sometimes the separation is effected by coarse filters instead of by sedimentation.

The amount of sludge that remains in the various processes of treatment cannot be closely approximated, but a rough indication of the comparative amounts can be found in Table XII.

TABLE XII

APPROXIMATE QUANTITY OF SLUDGE, DAILY, PER 1,000 PERSONS

Method of Treatment	Amount of Sludge Pounds
Broad irrigation	None
Intermittent filtration	None
Sedimentation and septic treatment	100 to 500
Chemical precipitation	1,000 to 4,000

Purification plants are generally arranged so as to use more than one process of purification, and often employ a combination of several processes. In most plants, the disposal of sludge is a serious problem; in some cases, it is disposed of by carrying to sea; in others, by burning (after pressing and drying); while in other plants, the sludge is disposed of by throwing it in trenches. Unsuccessful attempts have been made to treat it by various processes with the purpose of producing a marketable product.

SUMMARY

101. Problem to be Solved.—It has been pointed out that the problem involved in sewage disposal is to get rid of a vast volume of water containing, both in suspension and in solution, a small amount of organic matter, and a small amount of inorganic matter. The water must sooner or later be turned into a watercourse, the drainage from filters always finding its way to some stream. The organic matter, unless given an opportunity to oxidize, becomes putrid and offensive, and may, even when apparently inoffensive, contain disease germs. How, then, shall this small amount of matter be intercepted thoroughly, cheaply, and inoffensively, so that the water can be safely discharged into a stream?

102. Solution by Dilution.—Direct discharge into streams, lakes, or oceans may be practiced, most cheaply, without offense, if the dilution is in a ratio of about 1 : 40 or greater, if the currents or tides carry away the organic matter so that no deposits are formed, and if no danger is to be feared from the spread of infectious diseases.

103. Solution by Chemical Precipitation.—The method of chemical precipitation is not satisfactory in any regard. It is expensive, costing 50 to 75 cents per head per year for treatment alone. It is not complete, since only the matter in suspension is affected, that in solution being left to pollute the stream. It leaves as a residue a great volume of concentrated organic matter, no satisfactory way of treating which has been discovered. Its only application is to the preliminary treatment of manufacturing wastes.

104. Solution by Broad Irrigation.—The method of broad irrigation gives satisfactory results from a sanitary standpoint; that is, it takes out the organic matter thoroughly and without offense. The process is, however, not adapted to the sewage of a large city, both because of the expense of acquiring sufficient land, and because of the expense of

maintenance. As a rough guide, 1 acre must be provided for from 40 to 120 persons, or about 1,000 acres for a city of 100,000 inhabitants, an amount practically impossible to obtain within reasonable distance of a large city. The method is of great usefulness for the disposal of the sewage of such institutions as asylums, reformatories, etc., where agricultural land is abundant, labor is free, and large crops of roots and succulent vegetables serve to reduce the cost of maintaining the institution.

105. Intermittent Filtration.—The method of intermittent filtration, like the one last mentioned, is satisfactory except for cost. It was shown that sewage, with no previous treatment, can be applied to soil of the proper texture at a rate of about 50,000 gallons per acre per day, or about 500 persons per acre. No vegetation is permitted, the soil receives all the sewage it can take, and the purification effected is more thorough than by any other process. Many small cities where sandy soil in the immediate vicinity is available use this method.

106. Preparatory Treatment.—Some kind of preparatory treatment is today recognized as economical. This treatment should remove as much of the matter in suspension and solution as possible. It should give an opportunity for the anaerobic bacteria to act on the organic matter, thereby liquefying it, and it should allow the mineral matter to settle out. There are three ways of accomplishing this, namely:

1. *Septic Tanks.*—This is the most promising, because it is the cheapest, method for works of large magnitude. This method, which is to allow the sewage to flow slowly through a tank, removes a large part of the total organic matter, and places what is left in such a condition as to be readily acted on later.

2. *Upward Filtration.*—This yields perhaps better results than the first, but is more expensive. The process consists in straining the sewage upwards through coarse sand or fine gravel, at the rate of about 400,000 gallons per acre per day.

3. *Coarse Contact Bed.*—This method is the first stage in the multiple-contact method, and the result of the treatment is practically that of the two just given.

107. Final Treatment.—After the preparatory treatment just described, the sewage may be finally treated in one of three ways; namely:

1. *On Natural Soil.*—Under favorable conditions of topography and soil texture, the sewage may go on soil in place at the rate of about 250,000 gallons per acre. But this presupposes unusual conditions.

2. *On Artificial Filters.*—The sand in this case is brought, often for miles, specially selected, graded, and arranged in filters through which the sewage, after preparatory treatment, will pass at the rate of about 500,000 gallons per acre. Fine coal has been used with great success for these filters.

3. *On Secondary Contact Beds.*—This method allows the sewage to stand for about 1 hour in the coarse bed, and then flow on to the fine or secondary bed, where it remains another hour, and then is discharged purified.

This last method is fundamentally wrong, because it does not recognize the necessity of continually providing air to the filter material. The second method, supplemented by a sprinkling distributor, to spray the sewage on the bed, is not only found in practice to be the most successful, but is the method that the theory and principles explained in this Section would suggest as the most reasonable and logical.

IRRIGATION

INTRODUCTION

1. In spite of the large areas that, in certain sections, are already irrigated for the purpose of cultivation, it may be affirmed that irrigation as a science is still in its infancy in the United States. There are some countries, notably India, where it has not only been practiced from a remote period, but where also its principles are well understood, and its operations are carried on in a systematic manner and frequently on a gigantic scale. In those countries, irrigation has passed beyond the experimental stage, and has taken its place as an established branch of agricultural science. In still other countries, such as Mexico, although irrigation is not practiced on the large scale nor in the systematic manner prevailing in India, yet its advantages are thoroughly understood and appreciated; and it is very interesting, in traveling through the semiarid agricultural districts of that country, to note the skilful manner in which the small cultivators have taken advantage of every little streamlet to store and distribute the few scanty drops of water necessary to secure their otherwise precarious crops.

2. Necessity of Water in Raising Crops.—While it is pretty generally understood that water is necessary to the growth of plants, and that vegetation thrives with moisture and languishes in drought, it is not, perhaps, so generally understood why this is the case. Briefly, the reasons are the following: Plants require a certain, and often a very considerable, proportion of water as one of their constituent parts. This water they absorb from the soil by means of

their roots. The soil must contain a certain percentage of moisture—perhaps from 5 to 10 per cent.—before it can yield up any to the plant. On the other hand, the plant is constantly giving off moisture by evaporation from the leaves, and when the amount thus lost exceeds the amount taken up by the roots, the plant droops and dies. Besides, plants cannot assimilate their food in the solid state, but only in a fluid condition. It is necessary, therefore, that the substances they take from the soil should be in solution, and that the soil should contain enough water to dissolve those substances.

3. **Natural Irrigation.**—In those regions of the United States that enjoy a normal rainfall, comprising, as a rough approximation, all the territory lying east of the ninety-seventh meridian, the natural precipitation is depended on exclusively to supply the necessary solvent for plant food. That this precipitation may satisfactorily accomplish the result, two conditions are evidently necessary: first, the annual amount must be sufficient, and second, it must be distributed at proper seasons, so as to be timely as regards the needs of vegetation.

In the regions enjoying an average yearly rainfall of 30 inches or more—which would be much more than sufficient for plant growth if falling opportunely—there may still be serious damage done by drought, even to the extent of losing certain crops, from the fact that rain is lacking just at the right time. Even in the most favored localities, the element of uncertainty introduces a most dangerous factor in all agricultural pursuits. On the other hand, serious loss is often sustained by excess of rain when not wanted.

4. **Artificial Irrigation.**—The amount of water actually needed for the growth of crops is relatively very small. As a rough general average, it may be stated that between 12 and 20 inches per year, spread at proper seasons over the duly prepared surface of the area under cultivation, is sufficient. In order that these conditions may obtain, it is necessary to have under control and in store a volume of

water sufficient to cover the entire cultivated surface to the depth of from 1 foot to 2 feet, allowing for loss by evaporation and other causes, and to have also facilities for spreading this water over the given area as wanted. It is clear that this makes it necessary to provide for a considerable excess over the amount absolutely needed for irrigation, in order to provide for losses. But given a sufficient quantity of water, and adequate means of controlling its use, artificial irrigation is superior to natural, in that it gives the proper quantity of water to the plant, at the proper season. Indeed, the claim seems justified that, instead of artificial irrigation being a substitute for rain, rain is an imperfect substitute for artificial irrigation.

5. Commercial Value of Irrigating System.—A study of the cost and possible returns of irrigation work is instructive and essential, in order that the financial possibilities of any projected irrigation system may be determined. In the last censuses of the United States, statistical tables have been compiled that furnish an excellent basis for estimating the value of irrigation systems. The results vary greatly according to the region of the country considered, the nature of the soil, and the kind of crop that may be grown. Returns from irrigation in Southern California and in Arizona, where valuable crops of grapes, oranges, and other citrous fruits mature luxuriantly, and where four or five crops of hay may be cut in a season, are far greater than in Montana and in Wyoming, where only hardy grains and hay can be grown, and only one crop of the latter is obtained in one season.

The average cost of construction of irrigation works in 1902 was $9.84 per acre irrigated. The value of crops grown and marketed averaged, for each acre-foot of water used, in Montana, $18.42; in Utah, $6.34; in Arizona, from $10 to $30; in Northern California, from $10 to $20; and in Southern California, from $50 to $240. The average size of an irrigated farm in 1902 was 71 acres. As to the extent of irrigation, there were 9,487,000 acres irrigated in the West

in 1902, the total cost of construction being $93,320,000. The water was distributed through 33,415 systems, which included about 59,300 miles of main canals.

The average first cost of water per acre, as shown by the census, was $7.80. The average value of the products from this land per acre was $14.87. The average annual rental paid for the water per acre was $.38. The average value of irrigated land per acre was $42.53. Before irrigation was introduced, the average value of the same land was from $2.50 to $5 per acre. This shows the value added to land by the mere addition of water.

6. Irrigation Only One Factor in Cultivation. Mere irrigation must not be exclusively depended on to render arid soil productive, although in any case it may cause a temporary fertility at the start. Applied alone, and injudiciously, it may even decrease the fertility. It is only one of several factors in the reclaiming of otherwise uncultivable soil. It must be combined with a proper selection of crops suited to the particular soil; with proper underdrainage, natural or artificial; with cultivation and mellowing of the soil; with mulching; and in many cases with fertilizing. In a word, every other resource of the agriculturist should be brought into action, just as would be done in ordinary farming when no artificial irrigation is practiced. A neglect of those precautions has, no doubt, often led to disappointment and loss of faith in irrigation.

WATER FOR IRRIGATION

QUANTITY AND QUALITY

7. Units of Volume Used in Measuring Water. For irrigation purposes, the volume of water may be measured in cubic feet. There is, however, another unit that is very commonly employed, and is very convenient; it is called the **acre-foot,** and is the amount of water that will cover to a depth of 1 foot a flat surface having an area of 1 acre. As an acre contains 43,560 square feet, an acre-foot is equivalent to 43,560 cubic feet. A unit often used in the Western American states is the miner's inch (see *Hydraulics*).

8. Duty of Water.—The **duty** of a water used for irrigating a given area is the relation between the quantity of water used and the area irrigated. If water is measured in acre-feet, and area is measured in acres, the duty is expressed in acres per acre-foot, and is obtained by dividing the area irrigated, in acres, by the number of acre-feet used. Thus, if 4 acre-feet can irrigate 1.75 acres, the duty is $1.75 \div 4 = .4375$ acre per acre-foot. Occasionally, the duty is expressed as so many acre-feet of water required per acre of irrigated area, but this is really the reciprocal of the duty, not the duty itself.

9. Sometimes, the duty of water is stated as so many acres to the cubic foot per second. In this method of measurement, the water used during the irrigation period is assumed to be flowing at a uniform rate; the number of cubic feet per second is therefore obtained by dividing the total number of cubic feet used during the irrigation period by the number of seconds in that period. If the number of acres irrigated is divided by the number of cubic feet of water used per second, the result will give the duty expressed

in acres per foot per second. This method of expressing duty, although common, is in many respects unsatisfactory, and is disapproved by the best authorities on irrigation.

10. Duty per Acre-Foot.—"Assuming an average depth of 4 inches of water as sufficient to thoroughly soak the soil, this is equivalent to $\frac{1}{3}$ acre-foot per acre. An average crop requires from two to four waterings per season. Assuming three as the mean, then, at the above rate, 1 acre-foot will be required per season to irrigate 1 acre. Practice, however, clearly indicates that this theoretic amount is too low. Experiments conducted in Wyoming indicate that 12 inches in depth for potatoes to 24 inches in depth for oats is sufficient to mature crops. In Idaho, the depth of water generally used is about 24 inches; while in Montana, from 15 to 18 inches is believed to be sufficient. These indicate volumes ranging from $1\frac{1}{4}$ acre-feet in Montana to 2 acre-feet in Wyoming and Idaho. Measurements on several canals in Colorado show that from 18 to 24 inches in depth of water is required, or from $1\frac{1}{2}$ to 2 acre-feet per acre. Experiments conducted by Mr. Samuel Fortier in Utah indicate that a depth of 24 inches is required for tomatoes, while potatoes yield abundantly with a depth of 17 inches, onions with a depth of 36 inches, strawberries with 27 inches, and orchards with 12 inches. In India, the average duty of water, figured from the volume entering the distributary head, has been found to vary from $\frac{1}{3}$ to $\frac{1}{2}$ acre per acre-foot.

"In estimating the duty of water stored in a reservoir, allowance must be made for the losses due to evaporation and absorption in conducting the water to the fields. As this averages 25 to 50 per cent., it follows that, where a duty of 1 acre per acre-foot is possible, $1\frac{1}{4}$ to $1\frac{1}{2}$ acre-feet per acre must be stored in the reservoir; and where $\frac{1}{2}$ acre per acre-foot is the duty, $2\frac{1}{2}$ to 3 acre-feet per acre must be stored."
—*Wilson.*

11. In order that all consumers may not demand at the same time the use of the water flowing in the canals, it is found necessary to apportion the service periods for each

irrigator through the 24 hours of each day and through the season in such a manner that there will be a system of rotation employed in opening the heads of the ditches of the various users. The best method of arranging the rotation is to divide the main canals into a number of sections and the various distributaries (distributing canals) into sections, and to allow the water to only some of the sections at a time.

12. Irrigating Periods and Quantity of Water Required.—The length of time during which water is applied to land in the course of a season is called the **irrigating period.** Each such period is subdivided into several **service** periods.

"In Colorado, alfalfa and clover are irrigated twice in a season, once in May and once in June, to a depth of 6 inches for each period; wheat and oats are irrigated twice, once in June and once in July, to a depth of 9 and 6 inches, respectively. Meadow or native hay requires considerably more water; there are usually two service periods, each of which lasts several days, the water being allowed to run in a small quantity during that time. The first is usually in May, and is about 2 inches in depth for a week; the second in July or August, of about the same amount. Since the application of water is generally followed by a temporary checking of the growth of the plant, the method preferred in the arid region seems to be to give thorough rather than many irrigations; in other words, to have two ample rather than four to six small services. In general, it may be stated that two or three service periods, varying in depth from 3 to 6 inches, are employed in Colorado, and that the irrigation period extends from May to September—123 days. In Utah, the practice seems to be to employ a much larger number of service periods—from three to five on grain crops, of 2 to 3 inches in depth each—the water running 12 to 15 hours per service period, and the irrigation period extending from June to August, inclusive. On vegetables, as many as six or ten service periods are employed, each lasting from 3 to 6 hours, during June to August, inclusive. The irrigating

period in the majority of Western states averages from April 15 to August 15, or about 120 days; while the service period varies from 3 to 15 hours in length, according to soil and crop, and there are from two to eight such service periods in an irrigating period. In India, there are from three to five service periods, making up an irrigating period of from 100 to 130 days' duration."—*Wilson.*

13. Quality of Water.—Evidently it is not necessary to make so close a scrutiny of the quality of the water used for irrigation as for a domestic supply. Indeed, some waters totally unfit for domestic use from the presence of a large amount of organic matter are thereby rendered peculiarly favorable for irrigation, owing to the fertilizing properties of the substances held in suspension. But, although this suspended material is frequently beneficial to the land, it becomes sometimes very troublesome, by obstructing channels and waterways, and filling up reservoirs, particularly when the entrained silt is composed of mineral substances.

A very important factor in the value of water for irrigation is its temperature. The warmth imparted by water of a relatively high temperature is of itself frequently sufficient to greatly stimulate plant growth.

14. Drainage Connected With Irrigation.—In order that the territory operated on may derive the full benefit of irrigation, it is necessary that facilities be afforded for the removal of the surplus water after the soil has been thoroughly saturated. No benefit is derived if the soil is allowed to become water-logged. It is necessary that the water applied should slowly pass through the ground, and not remain on it until removed by evaporation.

Drainage, like irrigation, may be either natural or artificial. Frequently, the character of the soil is such that the drainage takes care of itself; this occurs when the ground is underlaid by a porous substratum; but at other times artificial drainage should be resorted to.

15. Drainage Necessary to Prevent Alkalinity. In many parts of the Western American states the presence

of "alkali" is a serious impediment to the growth of crops. The presence of alkali is manifested by a white efflorescence on the surface of the ground, consisting chiefly of chloride of sodium (common salt), sodium carbonate (sal soda), and sulphate of sodium (Glauber's salt). The effect of these salts on vegetation is most pernicious, particularly the sodium carbonate, known as "black alkali." The deposit of alkali on the surface of the ground is due to the evaporation of considerable quantities of water containing these salts in solution.

The best preventive of the formation of alkali is found in underdraining the soil. In regions where alkali prevails, soils not naturally underdrained should, if possible, be avoided, and only those that have natural advantages in this respect should be selected for irrigation. If the difficulty cannot be avoided in this way, it must be combated by artificial drainage.

16. Other Remedies for Alkali.—Although underdraining is the most radical and effective means of combating alkali, there are other remedies that may be employed, either alone or in connection with drainage. Mulching the soil, or giving it a top dressing of any kind suitable to shelter it and impede evaporation, is sometimes a valuable aid. The evil effects of black alkali are greatly diminished by the use of gypsum as a top dressing, but it appears to be thoroughly effective only when the soil is also underdrained. Sometimes, when there is an abundance of irrigating water, the deposit may be washed off the surface by flooding it, and rapidly drawing off the water before it can soak into the ground.

Some crops are less injured than others by alkali. Alfalfa, or lucerne, seems to be the least affected by it, and can be grown to advantage when other crops would fail.

It general it may be said that, in order to cultivate successfully ground afflicted with alkali, recourse should be had to underdraining, the use of a minimum amount of water in irrigating, cultivation, and mulching, and the application of plaster of Paris, or gypsum.

17. Silt and Sedimentation.—The rivers of arid regions carry in suspension, especially during flood times, great volumes of silt. This is derived chiefly from the erosion of the alluvial banks of the stream. As this enters the canals, the velocities of which are slower than those of the rivers, much of the suspended matter is deposited near the heads. The same is true when the matter enters the upper ends of storage reservoirs. The result is to diminish the volume of the reservoir, or the discharging capacity of the canal, as the case may be. These facts must be taken into consideration, therefore, in designing canals, in order that they may be given such grades that they will retain the finer silt in suspension until it reaches the land, where it is valuable because of its fertilizing properties. Also, canals should be so designed that as large a portion of the silt as possible shall be deposited within a short distance of the head, where provision may be made for removing it by dredging or by scouring.

Some of the Western streams carry vast amounts of sediment in suspension. On the Rio Grande it has been found to be as high as from one-fourth to one-half of 1 per cent. of the volume of the flow. On the American River in California, a depth of nearly 10 feet of wet silt was deposited in the reservoir in 1 year. In 12 years, 900 acre-feet of sediment was deposited in the Sweetwater Reservoir, California. On the Gila River, Arizona, sediment carried in suspension averaged about 10 per cent., and the amount of solids 2 per cent.

Silt-laden water has a very high fertilizing value. In the Moselle valley of France, barren land that was absolutely worthless without fertilization produced two excellent crops after irrigation with water heavily laden with alluvial matter. The turbid waters of the River Durance in France bring prices many times greater than that paid for the clear, cold water of some neighboring rivers.

SOURCES OF SUPPLY

18. There are two sources of supply that are commonly looked for in studying an irrigation project; namely, surface and ground waters. Generally speaking, all that has been said on this subject in *Water Supply* applies to irrigation. There are some points of difference, however, that must be noted. In the first place, as has already been mentioned, the question of hygienic quality is virtually eliminated from the problem. The chemical character of the water has, it is true, some bearing on its fitness for irrigating purposes, as being favorable or the reverse to the formation of alkali; but, broadly speaking, neither biological nor chemical examination plays any prominent part in this branch of hydraulic engineering. In the second place, since irrigation is mostly practiced in districts where the rainfall is abnormally small, general rules are less applicable, as regards the supply derivable per square mile of drainage area, than for districts of average rainfall and evaporation. More attention must be paid and more weight given to gauging, measuring, and observing, at least until a good general knowledge has been obtained of average conditions in the arid and semiarid districts that form the principal field of irrigating operations; and more pains must be taken to secure accurate results, on account of the small quantities dealt with. Each case will be more or less a special one, requiring special study.

SURFACE WATER

19. Preliminary Observations.—The first question to be decided will be the amount of water required. In this estimate, it will be well to make very liberal allowances for losses by waste, by evaporation, and in transmission. Suppose that in a given project it is thought that a yearly quantity of water, sufficient to cover the whole area to be irrigated to a depth of 24 inches, is necessary to include all items of use and loss. Then, if the given area contains 10 square miles, or 278,784,000 square feet, the yearly

amount of water required, in cubic feet, is this area multiplied by 2, or 557,568,000 cubic feet. If, therefore, it is desired to secure this amount of water by the aid of a storage reservoir, formed by building a dam across a certain stream, it is necessary to ascertain if the drainage area situated above the proposed dam, combined with the minimum available rainfall, is sufficient to afford the required quantity. It may be here remarked that, in the study of the quantities necessary for irrigation, the engineer is not bound so rigidly as in cases of water supply for communities. In the latter case, any failure in the daily quantity of water furnished leads to dangerous, or, at least, very inconvenient, results. Obviously, the failure to supply the full quantity that would be required for irrigation cannot be followed by so serious consequences as will a water famine in a populous city.

It has been explained in *Water Supply* that the available yield of any particular watershed or drainage area is not given by its area multiplied by the depth of yearly precipitation. A large percentage of this amount is lost by evaporation, by absorption, and from other causes. The remainder, which finds its way to the stream to which the drainage area is tributary, and which is known as the **run-off**, is all that is available for storage.

20. Study of Watershed.—The first thing that must be done in the study of an irrigation project derived from surface water is to collect data. These will consist of a survey of the watershed, gauging the flow of the stream, and measuring the precipitation or rainfall. The survey will be conducted on the same principles as the survey for any water supply. As from its nature it must be an approximate one, no time should be wasted in unnecessary refinement of instrumental work. A plain chain-and-compass survey is all that is needed.

The traverse survey should follow the crest of the drainage basin tributary to the reservoir site, in order that it may develop as nearly as possible the area of the catchment basin. Such a survey is unnecessary if the topographic maps of the

United States Geological Survey cover the area under consideration, in which case the area can be measured from those maps with all necessary accuracy. In like manner, the records of the United States Weather Bureau should be consulted for rainfall data, thus avoiding the necessity of making measurements. Finally, valuable data for determining the discharge of the water can be obtained from the records of the hydrographers of the United States Geological Survey, which may show that the stream itself or some of its neighboring tributaries draining like areas have been gauged.

21. Rainfall.—The subject of rainfall is fully treated in *Water Supply*, Part 1, where methods are given for gauging rain precipitation. Here, only that part of the subject that bears directly on irrigation will be considered.

The amount of rainfall is an extremely variable quantity: it varies not only with the locality, but also with the seasons, and often it is greatly different in two regions that are not far from each other. In the Western American states, it is much less in the flat and arid plains than on the high and well-wooded mountains that may separate them. In and near Sacramento, California, it is 15 inches per year, while at a short distance east, on the crest of the Sierras, it averages 50 to 60 inches. Still a little farther east, but at considerable altitude, in Nevada, the precipitation is as low as 5 to 10 inches. Near Yuma, Arizona, the precipitation during the growing season is only about 1 inch per year, yet not far off, near Prescott, it is 8 inches. The precipitation has been found to increase with altitude in the Sierras of California at the rate of about 6 inches per thousand feet increase in altitude. In the upper Missouri and Yellowstone valleys in Montana, the average annual precipitation is from 12 to 20 inches. Yet, of this amount, only about 5 inches falls during the growing season. In the Platte and Arkansas valleys of Colorado, as much as 8 inches falls during the irrigation season out of an annual precipitation of 15 inches.

22. The average annual precipitation is but an imperfect guide in determining the amount of water available for irrigation, if the water is to be used as fast, or nearly as fast, as obtained. Under such circumstances, the important factor is not the average precipitation, but the precipitation during the season of crop growth. The average annual amount over the northern Pacific coast would produce good crops, if it fell during the irrigating season, but the distribution of the precipitation is such as to materially affect its value for this purpose.

In connection with the water available for storage, however, the average yearly precipitation is of importance. Knowing the average amount of precipitation over the area of a basin tributary to a storage reservoir, it is possible to estimate with some degree of approximation the total volume of water available.

23. Exceptional conditions, such as heavy showers and snow melting, must be very carefully considered. Sudden melting of snow in the mountains after warm rains may produce disastrous floods. For example, the average rainfall near Yuma, Arizona, is but 3 inches; yet, single rainfalls of such violence as to amount to 2½ inches in 24 hours have been recorded. In San Diego, California, where the average rainfall is 12 inches, a storm amounting to 13 inches in 23 hours and aggregating 23½ inches in 54 hours has been recorded. Such storms may fill reservoirs to overflowing, cause the destruction of the dams, and ruin the irrigation system. Salt River, Arizona, has an average annual discharge of about 1,000 feet per second. Its average flood discharge is about 10,000 feet per second; yet, one storm is on record that caused a discharge of 300,000 feet per second, and of sufficient violence to wash away bridges and the heads of canals.

Another important condition is the suddenness of great rainfalls. Storms have been recorded with precipitation as high as 5 inches in 1 hour. In Baltimore, Maryland, a storm is on record lasting for 2 hours with a rate exceeding 6 inches per hour. The destructive effects of such storms

cannot be overestimated, and must be considered in the construction of water-supply systems.

24. Run-Off.—While the average and the periodical amount of precipitation are important factors in the designing of an irrigation system, the most important factor in determining the water supply available from any watershed is the *run-off*. As stated in *Water Supply*, the **run-off** is the quantity of water that flows off in a given time from the surface of the land. It is determined by gauging the flow in streams, and includes not only the water that flows from the surface after rainfall, but also that derived from springs, etc.

The amounts of run-off on various catchment basins (watersheds) in the arid regions of the United States have been determined and published by the hydrographers of the United States Geological Survey, whose publications on the subject contain a great deal of valuable information, and should, whenever possible, be consulted for the determination of the run-off of any watershed that it is proposed to use for irrigation purposes.

The run-off has been found to vary between .26 foot per second per square mile per year on the headwaters of the Gila River, Arizona, to .06 foot per second per square mile per year in the flat lands near its mouth. It ranges all the way up to 2.18 feet per second on the steeper slopes of the streams of the Sierras of California.

25. Gauging the Flow of Streams.—Streams are generally gauged by the erection of a dam, in which an overflow weir is placed, as described in *Hydraulics*. In some cases, the construction of a dam and weir is somewhat difficult, because the dam must be made water-tight, so that the entire flow of the stream will pass over the weir.

When great accuracy in the quantity of discharge is not required, or where the conditions under which the observations are made—such as possible leakage around the dam, inaccuracies of measurement, or uncertainty in regard to the velocity of approach—make the result doubtful, the following formula, in which a mean value of the coefficient of discharge

has been used, is simpler than the more accurate formulas given in *Hydraulics:*

$$Q = \frac{10}{3} l H^{\frac{3}{2}}$$

Daily observations should be recorded for at least a year, in order to obtain even an approximate knowledge of the normal flow of the stream. The time required to get an intelligent idea of the run-off and general character of a given watershed, by the above observations and those of the rainfall, constitute an embarrassing delay in districts where such observations have not already been carried on for a considerable period before it is desired to begin work. Unfortunately, it is very seldom that systematic records are kept in advance of the requirements.

26. Gauging the Flow With Current Meter.—Although the method of weir measurements is the best adapted to continuous observations, there are many cases in which its use is impracticable, or at least too difficult and expensive, and in such cases one of the other methods described in *Hydraulics* must be used. Of these methods, the one by the current meter is one of the best. With the improvements recently introduced by the engineers of the United States Army and the United States Geological Survey, this instrument gives results almost as accurate as those obtained by weir measurements.

In order that current-meter measurements may be made with accuracy, a straight and smooth bedded stretch of the stream must be selected, one in which there is little change in the slope or form of the bed for a distance of at least 100 feet above and below the gauging station. The cross-section of the stream must be carefully measured, the depth being taken at least every 10 feet. Daily observations should be made for a long period—a year or longer, if possible—so as to obtain good average values. The meter should be read at several depths for each of the different stages of the stream.

27. Measurement of Evaporation.—In the dry regions, where irrigation is mostly practiced, loss by evaporation is often a serious matter, especially for calculating

the proper capacity of storage reservoirs. In sections enjoying an abundant rainfall, this item of loss is rarely given much attention, since it is admitted that the rainfall on the surface of a reservoir will compensate for the loss by evaporation. Against this view, however, must be set off the fact that during the rainy season the reservoir is likely to be overflowing, when the benefit of any compensating precipitation is consequently lost.

It is very difficult to estimate in advance the probable loss by evaporation in a storage reservoir, because any experiments carried on in the stream that it is proposed to dam will be conducted under conditions different from those that will obtain in the reservoir itself. Evaporation is greatest in warm weather and during the prevalence of high winds. It varies with the character of the body of water from which it proceeds, as it is less in a deep reservoir than in a shallow one, and less in still than in running water. In the semiarid regions, evaporation will probably average from 3 to 5 feet in depth from the surface of a reservoir during the year.

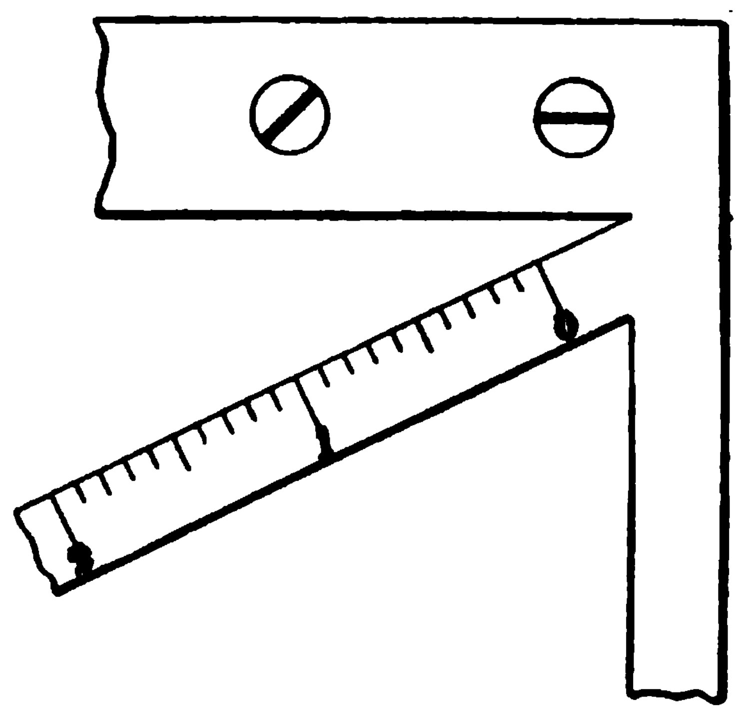


FIG. 1

To measure the evaporation from a given body of water, a simple apparatus is used by the United States Geological Survey. It consists of a pan of galvanized iron, 36 inches square and 10 inches deep, floated in the body of water of which the evaporation is to be measured, and filled with water to within a few inches of the top, care being taken to prevent water from washing in or out of it. A scale, Fig. 1, is placed obliquely in the pan, so that small vertical distances may be rendered appreciable by exaggeration.

28. Evaporation in any locality may be determined as above described. The records of measurement of evaporation

by the hydrographers of the United States Geological Survey and others should be consulted when possible, as they may furnish more valuable data than could be obtained by measurements extending over a limited period of time. In the arid regions of the United States, the monthly evaporation has been measured in many places, and is found to have a wide range. At Salt Lake City, Utah, it amounts to 40 inches per year and ranges from 1 inch in January to 7 inches in August. Near El Paso, Texas, the annual evaporation is 92 inches, ranging from 2½ inches in January to 1 foot in July. There is a perceptible though small amount of evaporation, about 6 inches, from snow and ice. In the arid regions, water is used for irrigation between the months of May and August, during which time evaporation is greatest, and as the precipitation is slight the losses sustained in this way are not compensated for by any corresponding rainfall.

29. Absorption.—The losses due to **absorption**—that is, to seepage and percolation from canals or reservoirs—are very considerable in amount, and must be added to those resulting from evaporation. In some localities, the losses by percolation have been found to be 25 per cent. of the rainfall, while the losses from evaporation were 75 per cent., the entire rainfall being thus lost. In sand, the losses by percolation are necessarily much greater than in earth or clay. By measurement, the losses due to absorption in a canal have been found to average about 1 cubic foot per second per linear mile. They are greater in new canals or in sandy soil and less in old canals, the beds of which have been made less pervious by deposition of sediment. In long canals, the losses by absorption in some years have been found to be as high as 60 or 70 per cent. Seepage into canals and reservoirs replaces to some extent the loss from evaporation and percolation.

GROUND WATER

30. Sources of Ground Water.—It has already been shown that a portion of the water that falls on the surface of the earth in the form of rain runs off rapidly into the rivers and streams, and is conveyed by them ultimately to the sea; while another portion sinks into the ground, and although this also is constantly seeking outlets and lower levels, by virtue of its weight, yet it moves so slowly, owing to the resistance to percolation offered by the media through which it passes, that at any given moment it may be considered as stationary.

During past ages, the water, constantly falling on the earth's surface and slowly sinking through deeper and deeper strata, has finally accomplished such a degree of saturation of the earth's crust that in almost all districts a permanent water-table has been established at a greater or less depth below the surface, and not varying very materially with the seasons. Even very arid districts, almost or wholly deprived of rainfall, may yet possess a store of underground water, which has slowly reached them from distant and more favored regions. We may thus consider the earth as forming a vast storage reservoir, in which the rainfall of many ages has been impounded. Should this supply be suddenly exhausted —were such a thing possible—it would doubtless require many centuries of subsequent rainfall to resaturate the earth's crust to the same degree that obtains at present.

The way in which this water is utilized by the construction of wells was explained in *Water Supply*. In some cases, ground water may be obtained by cutting tunnels intercepting subterranean streams, or by building diaphragms or dams across such streams, so as to force the water to the surface.

31. Pumping From Wells—Windmills.—Water is raised from wells by pumping, as explained in *Water Supply*. If the water is to be delivered above the level of the well, a force main is necessary. When the water from only a

single well is to be pumped and forced through a main, the problem offers no difficulty; when, however, a "gang" of wells is to be pumped, the problem is rendered more complicated. If the water rises to near the surface, say within 20 feet, and is not subject to fluctuations materially increasing this depth, all the wells may be connected with one general suction pipe operated by a single pump. Otherwise, a central station will be required, furnishing power for as many lifting pumps as there are wells, which will deliver their water to a common suction main.

32. The best power to work the pumps depends on circumstances. Whenever a constant water-power can be obtained, it will always be preferred, on account of its great economy. The principal objection to water-power is its liability to fluctuations.

Steam will be found, generally speaking, the most trustworthy source of power, and in some cases the engines and boilers may be advantageously located near the point where fuel is most cheaply procurable and where the power may be transmitted by electricity to the pumping station.

In the Far West, where transportation rates are high and all kinds of fuel must be brought long distances, gasoline is found to be one of the most economic fuels, and gasoline engines are in great favor for pumping.

More recently, electric power has been found most economical as a pumping agency, especially where it may be generated from water-powers located at reservoir sites or on running streams.

33. Windmills are employed for works of small magnitude; they are cheap, and give good results when properly used. There are many varieties of windmills on the market. The work that they will perform depends on the force and steadiness of the wind and the size of the wind wheel. In order to give security to the water supply provided by pumping with so erratic a motive power, sufficient storage capacity must be provided to hold the water pumped during the night time, and at such other times as it may not be used in

irrigation. The surplus water may be stored in one of the wooden or metal water tanks supplied by windmill makers, or in a reservoir excavated in high land and suitably lined with wood or cement, or clay and gravel.

The cost of operating windmills for a 25-foot lift, including interest charges, etc., ranges from 7 cents per hour for a 10-foot wheel to 24 cents for a 16-foot wheel, and 43 cents for a 25-foot wheel. A windmill is about 50 per cent. cheaper than a steam pump, but is relatively more expensive to install because of the necessity of providing storage. Windmills cost from $50 to $400, according to the size of mill and type of pump. The larger pumps will discharge from 250 cubic feet per hour for a 5-inch pump to 650 cubic feet for an 8-inch pump. A 12-foot steel mill will furnish about 1 horsepower in a 20-mile wind, and a 16-foot mill will furnish 1½ horsepower in a 20-mile wind.

WATER STORAGE

RESERVOIRS

34. Storage Reservoirs.—Having decided on the quantity of water required and the stream to be used, it is next in order to consider how much of the yield of the stream must be impounded in the storage reservoir.

The general subject of storage reservoirs has been treated in *Water Supply*. Owing, however, to the somewhat different services for which they are intended, there will doubtless be some difference between the functions of a storage reservoir for irrigation purposes and one intended for a public water supply. The draft made on the former will vary from year to year according to the wetness or dryness of the seasons, while the annual supply that must be furnished by the latter will be nearly constant. Irrigation reservoirs will also be depended on to furnish large quantities of water during short periods of time, thus making it necessary that they should be equipped with appliances for

meeting this heavy demand. With public water-supply reservoirs, on the contrary, the demand varies but little from day to day; hence, it is not necessary to provide them with capacious outlets, except such as may be required, in case of an emergency, to empty them quickly.

35. Location of Storage Reservoirs.—If, for distributing purposes, the storage reservoir is to be connected directly with the canal, flume, or pipe line by which the water is distributed to the territory to be irrigated, it should be placed as near as possible to the district to be supplied, so as to diminish the length of the canal. This plan also reduces considerably the losses due to percolation and evaporation. If possible, the reservoir should be so near the land to be irrigated that the water can be conducted through the entire distance in an artificial channel, in order to reduce percolation losses to a minimum. It may, however, be necessary to discharge the water into a natural drainage channel, and divert it therefrom lower down by a weir and canals. In such an event, the loss by evaporation and absorption will be much greater than when the water flows in an artificial channel having the most favorable cross-section. In the plains of the arid regions, natural lake basins can sometimes be utilized as storage reservoirs, which may then be filled by a canal diverting water from some near-by stream. Sometimes, a trial line of levels or a reconnaissance of the ground must be made to determine whether a proposed reservoir site is far enough above the highest point to be irrigated for the water to reach that point by gravity.

36. After the site has been selected, it should be very carefully surveyed, a map being constructed with contours having an interval of not more than 10 feet, so that the capacity of the reservoir may be determined for any given height. A careful survey must be made of the site of the dam, including borings to determine the nature of the foundation and whether or not the surface is underlaid with porous strata that will lead off the stored water. From such a survey, the exact cross-section of the dam, its cubic contents in masonry

or earth, and consequently its cost, and other engineering data may be determined.

37. Reservoirs for Wells.—It is seldom that the daily yield of a well is sufficient for the requirements of the seasons during which irrigation is practiced. It is generally necessary to provide a reservoir capable of containing a certain reserve that may be drawn on as wanted. If possible, such a reservoir is formed by building a dam across the valley of some stream, by which a combination may be effected, the dam catching whatever flow may come down the stream in the rainy season, as well as the supply pumped up from the well. In the case of springs, the reservoir is sometimes formed around the spring, which feeds it as a fountain does its basin.

DAMS

38. Timber Dams.—In works for irrigation, there will be more frequent occasion to build small, cheap reservoirs than in the case of a public water supply, because such works are often erected by individuals for private use on their own farms. Dams for reservoirs of this kind are usually constructed without the aid of a hydraulic engineer, but there is a right and a wrong way of executing even these small undertakings, and a few words about them will not be out of place.

These dams usually have a limiting height of about 10 feet, and impound not more than 1,000,000 cubic feet of water. Beyond these limits, the danger incurred by imperfect work becomes so great that nothing but the most substantial and scientifically constructed work, such as has been described in *Dams*, should be allowed.

The class of work now under consideration may be built on any one of many designs, which it would be impossible to consider in detail. Briefly, all these designs have for their object the avoiding of stone masonry laid up in cement mortar, which would place the structure in a higher class and consequently require skilled labor for its construction.

The best type for these home-made dams consists of timber cribwork, filled with well-packed stone, and backed on the

water side by a well-constructed earthen embankment. Loose stones not confined in cribs should be avoided, as they are liable to be carried away by freshets. The cribwork acts as a substitute for mortar, by binding the stones together so that they will act as a whole. Fig. 2 is a section showing the general features and minimum dimensions of a dam of this kind; the dam is 10 feet high to the level of the spillway or overflow. This figure, however, does not show the spillway, which is illustrated in Fig. 3 and explained later. To build such a dam, a good trench is excavated, deep enough to reach a satisfactory foundation, and a pavement of large stones, well-packed in with spalls, is carefully placed on the bottom. On these stones, the cribwork is placed, consisting of

FIG. 2

either round or square timber notched and treenailed together. The cribs should be carried well into the bank on each side, and the ends very carefully packed, to prevent water from passing around them.

The cribs are filled with stone of various sizes, tightly packed. The face of the crib on the up-stream side is sheathed with tight planking. Against this sheathing the earthen embankment is placed, the natural surface of the ground beneath it having been roughly excavated, by plowing and scraping, for a foot or so, according to the nature of the soil. This embankment should be riprapped.

Fig. 3

The spillway for these dams should be ample; the length may be about 10 per cent. greater than for the carefully built dams described in *Dams*. At the spillway, the cribs require reinforcing; this is accomplished by extending and stepping them, as shown in Fig. 3. The face timbers should preferably be made square, for the convenience of spiking on the planking with which the face should be completely sheathed.

It must be repeated that Fig. 2 shows minimum dimensions for first-class work of its kind. For ordinary work, it

will be prudent to somewhat exceed them. The thickness of the solid stone and timber work should never be less than the maximum height to which the water may rise behind the dam.

Such dams, when care and judgment are used in their construction, are safe and substantial so long as the timber forming the cribs remain sound. Under ordinary conditions, it will be long before the timber decays. If the work is well done and good material is used, the embankment will in time become so packed and consolidated that, even after the cribs have, through decay, lost much of their original strength, the dam will continue to be a safe structure. Dams, however, of this construction are not to be considered permanent in the same sense as stone dams or even earthen dams with a masonry center wall.

Probably the simplest and best way to draw off the water from a dam of the type just described is by means of a cast-iron pipe running through it and under the embankment. The pipe should be placed, if possible, on the natural surface of the ground, to avoid settlement. If this is impossible, a stone foundation resting on the natural ground should be built under it. Gates or stop-cocks should be placed in the pipe, outside of the dam. Sluice gates, stop-plank, etc. will not be needed in this class of work, if the above pipes and appliances are used.

39. Reinforced-Concrete Dams.—A noteworthy, because comparatively recent, deviation from the usual type of masonry dam is one built of concrete with embedded steel to reinforce it. Temperature changes in high masonry dams cause expansion and contraction, especially in the part above water, thus producing stresses that may rupture the structure. To guard against such rupture, which is usually the result of tensile stress, iron or steel, chiefly railroad rails, is embedded in the upper portion of the dam. Some excellent dams of moderate height have been built of reinforced concrete throughout. By the use of this material, it has been possible to give a slighter cross-section, and in some

cases material is saved by making the dam a hollow shell of concrete heavily reinforced with steel bars.

40. Loose-Rock Dams.—In inaccessible portions of the arid regions, where the only foundation available is rock or hard pan, and where rock is abundant near-by and earth for construction of earth dams is not accessible, some substantial structures have been built of loose rock. The loose-rock type may be especially desirable where the cost of cement is high. Such a structure consists of masses of broken stone, just as it is blasted out of near-by hillsides, the smaller stones being used to fill up the spaces between the larger ones. Side slopes of about 1 to 1 are usual. The top width should be 10 to 20 feet, according to the height, and an ample wasteway should be provided as for earth dams, to avoid the possibility of the crest being topped by overflowing water. One advantage of loose-rock dams is that they may be constructed in flowing water. Another is that a leak is not so serious a menace as in earth or masonry dams.

In building a loose-rock dam, the material should be deposited in layers in such a manner that the extremities of each layer shall be higher than the center. The upper slope should be faced with planks or with iron plates to render it water-tight. To prevent seepage through the dam, a center wall, having the same position as the core of an earthen dam, is usually built; this wall may be made of rubble masonry, of plain concrete, or of boiler plates embedded in concrete.

CONDUITS

41. When a large volume of water is to be conveyed a long distance and distributed in measured quantities over an extensive territory along the route, open canals having a suitable fall are generally employed. Such canals have some objectionable features, one of which is, a considerable loss by evaporation and percolation, the latter being an unknown quantity until the canal is built and put in operation. They must also follow a nearly uniform and easy grade, a condition that makes it necessary either to skirt along the valleys they encounter, thereby greatly increasing their length, or to cross these valleys on aqueducts, which are always more or less expensive structures.

When the volume of water to be conveyed is not very great, canals can be replaced by flumes or by pipe lines. Flumes, being open channels of wood, iron, or concrete, differ from canals chiefly in the material of which they are made. Pipe lines, which may be of either cast-iron, wrought-iron, steel, or wooden-stave pipe, differ from canals and flumes in the fact that they are not confined to a uniform descending grade, but may go up or down hill, and, if necessary, can be laid in the bed of any stream that they must cross.

These classes of conduits will be taken up in order, beginning with open canals.

CANALS

LOCATION AND GENERAL DESIGN

42. Preliminary Surveys.—Surveys of a more or less extensive character are the necessary preliminaries to the design and construction of either a canal or a pipe line; this is especially true of the former, because the course of the canal is confined to narrower limits than that of the pipe

line. Although much of what follows regarding surveys will be common to both, it must be understood throughout that it is a canal line that is especially under consideration.

Such a line naturally begins at some point very near to the stream that furnishes the water, and it will generally follow the same valley for a considerable distance. If time permits, it will be of the highest utility to secure a general survey and profile of the stream itself for the entire distance that the canal follows it, noting all tributaries, falls, and rapids. It will be found that it pays well to make a careful and thorough survey, as more work can frequently be done in a few days with transit, chain, and level than in many weeks with pick and shovel. As far as alinement is concerned, this survey does not call for any great accuracy. The leveling, however, is of much importance; for it must be remembered that in this work very large errors are likely to pass unperceived, and that no line of levels can be trusted that has not been checked. Therefore, when the alinement has been completed and leveled, check-levels should be run back over the entire line; it will not be necessary, however, to verify the entire profile, a check on the benches being sufficient. All important tributaries should also be surveyed, the survey being carried up the valley until an elevation approximately equal to that of the starting point has been reached.

A rough estimate of the length of the canal can be made from this survey, in connection with the topographical notes taken at the same time; then the approximate length, together with the total fall to be overcome, will enable the engineer to make a preliminary design of the section and grade.

43. Trial-Line Location.—A trial line for the canal can now be run. There are several ways in which this can be done. One of the best and most expeditious methods is as follows: Suppose that a grade of 4 feet to the mile is decided on for the slope of the canal. The tangent of the angle corresponding to this slope is $\frac{4}{5280}$ = .00076, which corresponds to an angle of nearly 3 minutes. Having

a transit provided with a vertical limb, the telescope is depressed to this angle, clamped, and turned in the approximate direction of the line to be followed. The target of a leveling rod is then set at the height of the telescope of the transit from the ground, which can be sufficiently approximated by holding the rod alongside of the transit and sighting across the wyes. The rod is now taken as far ahead as possible and moved along the ground, up or down hill. At the same time that the rod is moved up and down, the transit may be slightly turned in azimuth to the right and left, until the horizontal cross-hair cuts the target. The foot of the rod is then on ground at the desired elevation; a hub is driven at this point, and the distance from the instrument is measured. The transit is then set up over this hub, and the operation repeated. The relative directions of the lines thus run may be ascertained by the needle, as this will be sufficiently accurate. From time to time, measurements will be taken to convenient stations on the line of the river survey, if such a survey has been made, as a check. It will be well to carry this line along, following all the indentations and tributary valleys. It will be very rare that this line is actually followed by the canal, as it would probably be too devious. Some valleys will be crossed on aqueducts, and some hills will be tunneled; but only by a complete survey can the comparative advantages of alternative lines be compared. When an approximate location of the line has thus been determined, the line should be accurately rerun and leveled over, so as to establish the final location and make a more nearly exact estimate of cost.

44. Grade.—The grade of the canal depends on several conditions. In the first place, the character of the bottom and sides will place certain limits on the velocity of the water, which must be great enough to prevent the deposition of silt, and not so great as to do injury to the canal bed. The grade necessary to maintain the velocity within the desired limits will also depend on the character of the interior surface of the canal, being very much less for a

surface having a smooth lining—of concrete, for instance—than for one merely excavated in the earth. The form and area of cross-section also affect the grade, because they affect the velocity of flow. In general, the grade depends on the velocity, and the latter must be determined first.

Formulas for the flow of water in channels are given in *Hydraulics*. Kutter's formula is applicable in all cases, although, for some special conditions, there are other formulas that are simpler and lead to sufficiently close results.

EARTHEN CANALS

45. Formula for Canals With Earthen Banks.—An approximate formula that may be used for canals with earthen banks in good condition is the following:

$$v = \sqrt{\frac{100{,}000\, r^2 s}{9r + 35}} \qquad (1)$$

in which v = mean velocity, in feet per second;

r = hydraulic radius, in feet, $= \frac{a}{p}$;

s = slope $= \frac{h}{l}$.

Solving formula **1** for s, we get, for the slope,

$$s = \frac{9r + 35}{100{,}000}\left(\frac{v}{r}\right)^2 \qquad (2)$$

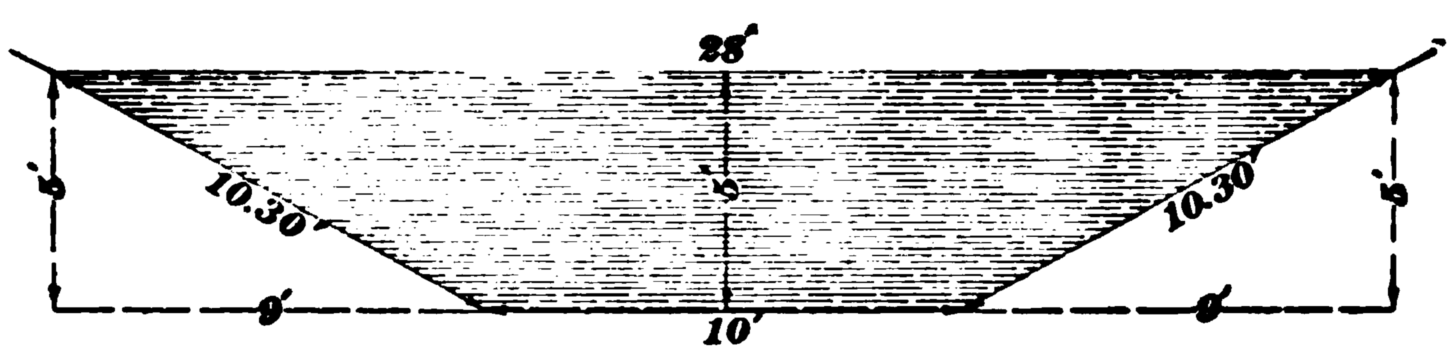


FIG. 4

EXAMPLE 1.—If the fall in an earthen canal having the cross-section shown in Fig. 4 is 5.25 feet per mile, what is the mean velocity of flow?

SOLUTION.—Here $s = \frac{5.25}{5{,}280} = .001$, nearly; also, $r = \frac{95}{30.6} = 3.105$. Then, by formula **1**,

$$v = \sqrt{\frac{100{,}000 \times 3.105^2 \times 0.001}{9 \times 3.105 + 35}} = 3.91 \text{ ft. per. sec.} \quad \text{Ans.}$$

Example 2.—With the same form and dimensions as before, what should the slope be, if the velocity is to be 2.5 feet per second?

Solution.—Substituting known values in formula **2**,

$$s = \frac{9 \times 3.105 + 35}{100,000}\left(\frac{2.5}{3.105}\right)^2 = .0004. \quad \text{Ans.}$$

46. Limiting Velocity.—It has been found that light and sandy soils cannot safely resist a mean velocity greater than 2 feet per second, while at the same time this velocity is sufficient to prevent plant growth and to remove silt. In firmer soils, velocities of 3 to 4 feet per second are permissible, but, except in hard pan or a material of considerable resistance, 5 feet seems to be the limiting velocity for earthen canals.

In almost any district where it is proposed to build such canals there will be some examples of ditching, on a greater or less scale, by observing which an approximate idea may be formed of the proper grade and side slopes to be given to the proposed canal. The engineer should not fail to take advantage of all such opportunities to obtain local knowledge of the district in which he is working.

47. Form and Dimensions of Cross-Section.—The cross-section of an earthen canal is customarily made in the form of a trapezoid. The side slope, or inclination of the sides to the vertical, is usually determined by the nature of the soil, and a certain depth will be found more convenient or desirable than another. By taking these and other points into consideration, the designing of a proper cross-section to satisfy the necessary requirements is simplified. The method of procedure will be best understood by an example.

Suppose that it is desired to establish the proper cross-section and grade of an earthen canal, under the following circumstances: The quantity of water to be conveyed is 250 cubic feet per second. A velocity of 2 feet per second is desired. The side slopes are to have an inclination of 1 vertical to $1\frac{1}{2}$ horizontal, and a depth of 6 feet of water is desired in the canal. What should be the form and area of the cross-section, and what the grade of the canal?

Since the velocity is to be 2 feet per second and the discharge 250 cubic feet per second, the area of cross-section must be $\frac{250}{2} = 125$ square feet.

To determine the dimensions of the cross-section, it is necessary to know the bottom width of the canal, which

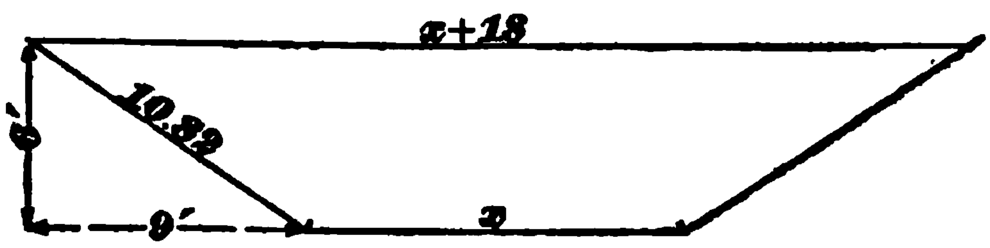


FIG. 5

is represented in Fig. 5 by x. From the data, we have

$$6 \times \frac{(x + x + 18)}{2} = 125;\ \text{whence, } x = 11.83 \text{ feet.}$$

Using a value of 12 feet, which gives the slightly greater area 126 square feet, the wetted perimeter is $10.82 + 10.82 + 12 = 33.64$ feet, and the hydraulic radius is $\frac{126}{33.64} = 3.75$ feet. Formula **2** of Art. **45** now gives

$$s = \frac{9 \times 3.75 + 35}{100{,}000}\left(\frac{2}{3.75}\right)^2 = .000196$$

This represents a grade per mile of $.000196 \times 5{,}280$, or 1.03 feet.

48. Effect of Depth on Slope.—The depth of the canal has a considerable effect on the velocity of flow. Thus, in the foregoing illustration, if a depth of 8 feet is used, all the other data remaining the same, and using the same area of 126 square feet, we shall have $8(x + 12) = 126$; $x = 3.75$.

The depth being 8 feet and the ratio of slope being 1 to $\frac{1}{2}$, the length of the side slope is 14.42 feet, and the wetted perimeter is 32.59 feet. Therefore, the hydraulic radius is $\frac{126}{32.59} = 3.87$. Then,

$$s = \frac{9 \times 3.87 + 35}{100{,}000}\left(\frac{2}{3.87}\right)^2 = .000187$$

This represents a grade of .99 foot to the mile, as against 1.03 feet for the previous depth.

49. General Remarks on Earthen Canals.—Earthen canals, particularly in light, sandy soils, often give a great deal of trouble, even when properly side-sloped and graded, by reason of the tendency to wash; their use is, therefore, mostly confined to those very large works where the use of pipes or flumes would be out of the question. When lined with masonry, they are much more efficient, and the greater velocity that they can then safely sustain, and their consequently greatly reduced cross-section, makes their relative expense, as compared with canals having unprotected interior surfaces, less than might be imagined. Sometimes, cheap substitutes for masonry lining are employed, and it has been found in California that a good and durable lining can be made by coating the sides and bottom with a plastering $\frac{3}{4}$ inch thick, composed of one part of Portland cement and four parts of sand, the sides and bottom having previously been accurately trimmed and moistened.

50. Earthen canals are best when built entirely in excavation. It is impossible, however, to obtain this result unless the ground is exceptionally favorable and the location very carefully selected. Even then such a canal will have a greatly increased length, owing to the necessity of many deviations, in order to keep it on suitable ground. Practically, for a large proportion of their length, canals will be formed partly in excavation and partly in embankment, the material thrown out of the excavation being used, if suitable, in the embankment. Great care must be taken to carefully trim the banks to true lines.

51. The proper inclination to give to the side slopes is a point requiring very careful consideration. Both very steep and very flat slopes lead to deterioration by wash. If too steep, they fall by the undermining effect of the flow of water in the canal; and if too flat, the exposed surfaces are damaged by rain. The best guide is a careful examination of any canals or ditches that may be found already in use in the district.

When the embankment is high, it is better to keep a narrow berm between the foot of the bank and the edge of the

ditch. When very high, a center wall should be used, as for earthen dams; this, of course, adds to the expense. Fig. 6 shows a half section in which these features have been carried out.

FIG. 6

52. It has been observed that in banks and excavations for both canals and railroads the effect of time and wash is always to reduce the original straight lines and sharp angles to curves and rounded edges. It would undoubtedly be an advantage to anticipate this result by giving to such work, at the start, a form somewhat similar to that which it will eventually assume. Thus, in Fig. 7, if the heavy straight

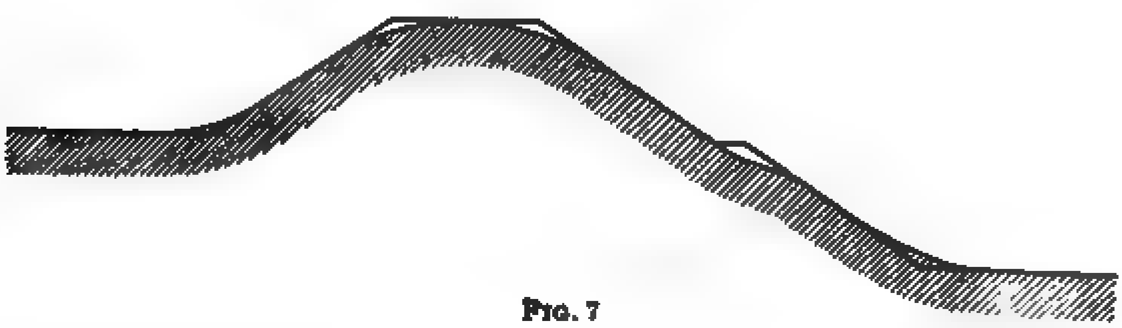


FIG. 7

lines represent the original form of the cross-section, it will gradually assume the shape shown in the shaded portions. It will be better, therefore, to favor this form in shaping the slopes of the excavation and embankment.

LINED CANALS

53. Canals Revetted With Dry Stone.—So much trouble is occasioned by the deterioration, slow or rapid, of canals with unprotected banks, and so much uncertainty

exists regarding their probable discharge, that it is often good policy and economy to line them at least with dry stone. Such an arrangement is shown in Fig. 8, which represents a canal cut in sloping ground. Any stones that can be obtained may be used for the purpose, but flat stones are preferable. It is generally best to lay the pavement continuously under the side walls, and to build these on it, as

FIG. 8

shown in the figure. Some rough hammer dressing is usually required at the corners, where the side walls connect with the pavement. In laying the pavement, if flat stones can be procured, they should all be laid on edge, with a slight inclination down stream, and packed as closely as possible. All the work should be thus packed and the walls well bonded. This lining can be much improved by pointing it with cement mortar.

54. Formula for Velocity of Flow in a Canal Lined With Dry Stone.—It is very difficult to derive a formula for canals lined with stone, because the velocity of flow depends, in a large degree, on the character of the lining and the quality of workmanship. The following approximate formula may be used for a well-laid dry wall, without pointing or plastering:

$$v = \sqrt{\frac{100{,}000\, r^2 s}{8\, r + 15}} \qquad (1)$$

Solving this equation for s, there results

$$s = \frac{8\, r + 15}{100{,}000}\left(\frac{v}{r}\right)^2 \qquad (2)$$

EXAMPLE.—Referring to Fig. 8, let the bottom width of a canal lined with dry stone be 8 feet, the batter of the side walls being 1 vertical to $\frac{1}{2}$ horizontal. Let the depth of water be 8 feet, and the desired velocity 7 feet per second. What is the value of s?

SOLUTION.—Here the breadth of the waterway at the surface of the water is 16 ft. The area is, therefore, $\frac{16+8}{2} \times 8 = 96$ sq. ft. The length of the wet line on a section of the side wall, with the given batter and depth of water, is $\sqrt{4^2+8^2} = 8.94$ ft., and the value of p, or the wetted perimeter, is, consequently, $2 \times 8.94 + 8 = 25.88$ ft. Therefore, $r = \frac{96}{25.88} = 3.71$. Substituting known values in formula **2**, we have

$$s = \frac{8 \times 3.71 + 15}{100{,}000}\left(\frac{7}{3.71}\right)^2 = .00160. \quad \text{Ans.}$$

55. Formula for Canals Lined With Rubble Masonry.—A canal lined with masonry laid in cement constitutes a still higher type of structure. It is far more permanent in its character than the canals already considered, permits of a higher velocity of flow without injury to itself, and with a given grade and hydraulic radius offers less resistance to rapid flow.

The following formulas apply to a canal lined on the sides and bottom with good rubble masonry, pointed but not plastered:

$$v = \sqrt{\frac{100{,}000\, r^2 s}{7.3\, r + 6}} \qquad (1)$$

$$s = \frac{7.3\, r + 6}{100{,}000}\left(\frac{v}{r}\right)^2 \qquad (2)$$

56. Concrete-Lined Canals.—Now that cement is so much cheaper than formerly, the practice of lining critical parts of canals with this material is very common. This is especially true of the great canals built in the arid regions of the United States by the Reclamation Service of the Geological Survey. This kind of lining is especially adapted to porous soils, through which it would be sometimes impossible to convey water, on account of the great loss caused by percolation.

A typical section of a concrete-lined canal is shown in Fig. 9. The lining should be 6 inches in thickness; on the

outside of curves, it should be made 1 foot higher than shown in the figure. In sections where velocities exceed 12 feet per

FIG. 9

second, the lining should be 7 inches in thickness. The flow in a canal of this kind can be computed by Kutter's formula, using a value of .012 for the coefficient of roughness.

EXAMPLES FOR PRACTICE

1. An earthen canal is to discharge 200 cubic feet of water per second, and the side slopes are to be 1 vertical to 1½ horizontal. The depth of the water in the canal being 5 feet and the velocity 1½ feet per second, what must be the value of *s*? Ans. .00012

2. The bottom width of an earthen canal is 8 feet, and the side slopes are 1 vertical to 1½ horizontal. If the depth of the water is 6 feet, and the slope is 5.25 feet to the mile, what is the discharge in cubic feet per second? Ans. 432.5 cu. ft.

3. The bottom width of a canal lined with dry stone is 9 feet; the batter of the side walls is 1 vertical to 1 horizontal. The depth of the water being 9 feet, what should be the value of *s*, in order that the velocity may be 8 feet per second? Ans. .0015

HEAD-WORKS FOR CANALS

57. General Description.—At the point where the water from a running stream is to be diverted into a canal, a group of structures, known as **head-works**, must be built. These consist of a weir or dam built across the stream for the purpose of forcing part of the water into the canal, together with another structure, called a **regulator**, built in

the canal head, and closed by gates designed to admit the water to the canal. At that end of the weir which is adjacent to the canal is a passage, usually open, but so arranged that it may be closed, in whole or in part, by gates; this passage permits, at high water, a free flow past the regulator, and thus acts as a sluice for scouring the channel of the stream. In the canal, at a short distance below the regulator, is constructed an escape for any surplus water that may have been admitted to the canal.

58. Weirs.—Weirs may be built of brush and boulder barriers, which are easily washed away and must be replaced after each high water. They may be made of rectangular sheet piling filled with rock, or of open woodwork closed with flash boards, or of loose rock, wooden cribwork, or masonry. The form to be selected depends on the permanence of the work, the character and size of the stream, and the materials available. A favorite type of wooden weir is a series of **A**-shaped frames placed across the channel and founded on piling. The upper surface of these frames may be permanently closed with planks securely fastened, or a portion of it may be closed with planks let into grooves, and may be removed when the water rises, or for scouring. In India, a type of weir constructed in rivers of great discharge consists of long, low, flat slopes, the lower slope being 1 in 20 or more, the upper slope 1 in 2 or 3, the height being but a few feet. Along the line of the crest is built a masonry wall, and the remainder of the weir is of loose rock. Masonry weirs may be founded on crib, on piling, or on solid rock. They are given various cross-sections, similar to those of the larger dams used for storage reservoirs. The lower slope may be ogee shaped, or the water may fall into a water cushion on the lower level.

Weirs for head-works have, in some cases, been built wholly of steel, a series of steel frames being erected across the stream channel, substantially bonded into the rock surface of the bed, and covered with plate iron. Of late years, reinforced concrete has, in a measure, superseded solid

masonry for the construction of such works, this form of structure being hollow instead of solid.

59. Scouring Sluices.—The **scouring sluices,** which may be placed in the end of the weir adjacent to the canal head, consist of a series of piers between which are gates that slide vertically and are operated by screw gearing or by a similar device. Sometimes, they are closed by shutters that act automatically; these shutters fall when the water in the stream reaches the danger height to which they are adjusted.

60. Regulators.—The **regulators** at the canal heads consist of a series of sluices similar to the scouring sluices. They control the admission of water to the canal, and when closed cause it to pass down the stream over the weir. The regulator should be so placed that the water held up by the crest of the weir will pass immediately through it into the canal, and should be so close to the weir that it will practically form a part of it. Moreover, the regulator should be so inclined that the water flowing past its head will scour the bed clean at that point and prevent the deposition of silt. Regulators may differ in character according to circumstances, and may consist of wooden gates in timber framing, wooden gates in masonry or steel framing, or steel gates in masonry or steel framing. For convenience in operating the various openings in the regulator, the piers are bridged by an overhead walk from which the gates are operated. To withstand the pressure of great floods, the regulator must be firmly and substantially constructed.

CONSTRUCTIVE DETAILS

61. Overflows and Emptying Sluices.—When a large body of water is conveyed in a flume or a canal, there is always a certain degree of danger to be apprehended from a possible overflow, due to a sudden diminution of the draft without a corresponding reduction of the supply. This may be occasioned either by a shutting off of some of the outlets,

or by some accidental obstruction occurring to arrest the flow. An overflow from any cause, unless directed through proper channels, is particularly dangerous in the case of an earthen canal, and precautions must be taken to render it impossible.

It is also necessary to provide means for emptying the canal at any time, without employing the ordinary outlets used for irrigation.

The first of these requirements must be provided for, in the case of earthen canals, by building **overflows**, or **spillways**, at intervals depending on the grade of the canal, each of which should be capable of safely discharging the maximum volume of water that can reach it, supposing the entrance gates to the canal to be wide open. These spillways should be constructed precisely as described for those used in dams, taking care that they are provided with substantial wing walls and aprons, to prevent scour and wash from the escaping water. Overflows are sometimes called **escapes**.

62. The appliances used for emptying a canal consist of sliding sluice gates. These may be elaborate in construction, such as metallic gates set in masonry chambers, or they may be somewhat rudely fashioned of timber. It will be observed that these gates may have the combined office of emptying the canal and preventing overflow, but it is not safe to depend on them for the latter purpose, because, through negligence, they may fail to be opened at the proper time, whereas the action of an overflow or spillway is always automatic. Overflows and sluice gates should be located at or near the crossing of a stream, or at least at some natural depression of the ground, in order to provide for an easy escape of the water.

63. In large canals carrying a heavy body of water, both overflows and sluice gates assume considerable proportions and call for great care and skill in their design and construction. For such structures, the plans of similar works should be carefully studied, and used as guides for any particular

case. It would be impossible to give instances covering all conditions.

The most rudimentary form of sluice gate will be one inserted in a timber flume at the crossing of a stream. It will consist subtantially of a sliding gate working in a groove between upright posts. It may be operated, if small, by a lever, otherwise by a rack and pinion.

64. Culverts.—When a canal crosses a stream or the natural drainage channel at about the same level, a culvert may have to be constructed for carrying the canal over or under the channel, as the case may be. Frequently, such culverts are built of wood, usually in the form of rectangular pipes founded on piling, where the material forming the valley bottom is not stable. Proper approach wings are made at the inlet and outlet ends, and sufficient dimensions are given to carry the full volume of the canal or the flood volume of the stream. Culverts of large dimensions are usually made of masonry or reinforced concrete. The latter material has been used in some of the larger canals designed in the arid regions of the United States. For a full treatment of the subject of culvert construction, see *Culverts*.

65. Drops.—A canal usually runs through country having a greater natural fall than the canal. To compensate for the difference between the slope of the country and that which may be safely given the canal bed in order that the water in it may have the desired velocity, the canal is divided into longitudinal sections having the proper slope and connecting at the ends by **drops**, which are either vertical falls or inclined rapids. The place for a drop is determined by the point at which the canal rises above the surface of the ground.

Drops may have a clear vertical fall to a wooden or masonry apron; or the lower face of the fall may be given an ogee-shaped curve; or, to diminish the erosive action, the water may be allowed to plunge into a water cushion. Above the fall, a weir crest may be built, or the channel may be contracted, or gratings may be introduced to diminish the scour.

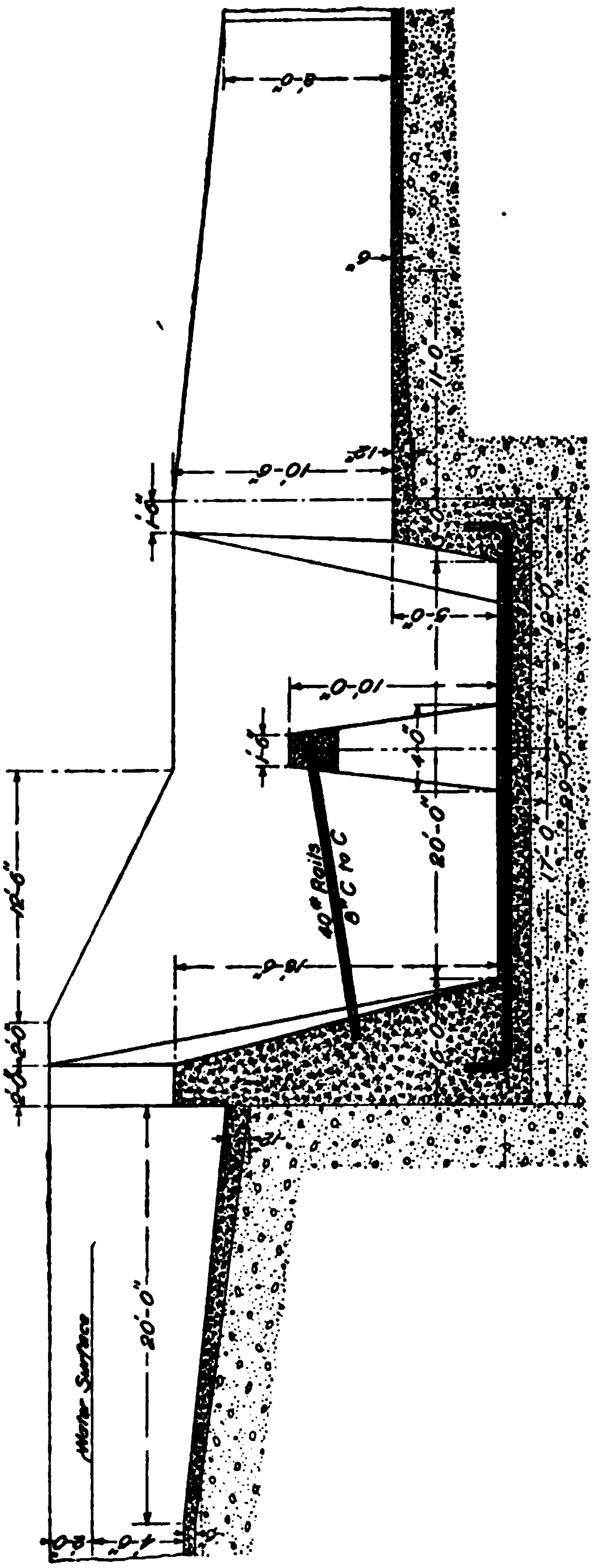


Fig. 10

136—35

66. In the past, the practice in the United States has been to build falls of wood, the bed of the stream being floored with planks and its sides protected by wings, both above and below the crest of the fall. In some cases, water cushions have been made below the crest by lining a depression in the lower channel with wood. In India and in Italy, the practice has been to construct falls of substantial masonry. The expense of such work has heretofore been considered prohibitive in the United States. Recently, in some of the drops designed by the Reclamation Service, the practice of protecting the bed and banks of drops by reinforced concrete has been adopted, and has been found to be most effective and not too expensive. An excellent example of such

a drop is that shown in longitudinal section in Fig. 10. The water from the upper canal level passes over a bed lined with 12 inches of concrete to the edge of a weir crest constructed as a masonry retaining wall 17 feet high and 2 feet wide at the top. Thence, it drops a height of 15 feet 6 inches into a water cushion 20 feet long and formed by a masonry retaining wall protecting the lower level of the canal. The height of this drop is so great that, to reduce erosive action, the force of the falling water is broken by a series of iron rails placed 8 inches apart between centers and at a height of about 7 feet above the floor of the drop. The ends of these rails are supported in some such manner as shown in the figure. The total depth of the water cushion is 5 feet, and this and the rails tend greatly to reduce the scouring action of the falling water. The concrete in the bed of the drop and in the walls lining it is heavily reinforced with iron bars running both longitudinally and transversely.

67. Turnouts.—Wherever a branch is to be diverted from a main canal, or where a canal is divided into two branches of nearly equal dimensions, it is necessary to locate a **turnout** for the diversion of the water from the main canal. These turnouts include some appliance for forcing a portion of the water from the main canal into the branch, in the head of which is an appliance for regulating the amount of water it is desired to admit. Turnouts may consist of wooden linings to the earth banks and bed of the canal, and be constructed somewhat after the fashion of a flume with wing walls properly supported by piling, etc.; or they may consist of masonry. More recently, the better forms have been made of reinforced concrete. The lining is carried up and down the canal and the branch a sufficient distance to protect the bed and bottom. In the canal are erected piers for the support of gates, or stops. These gates usually consist of planks working in grooves, with which the piers are provided, and serve to dam the water and force part of it into the branch. At the inlet to the branch, one or more gates, operated usually by wheel and gearing, are placed to admit

the proper amount of inflow. Sometimes, where but small volumes are diverted into minor distributaries, the turnout may consist of pipes or culverts laid through the banks of the main canal and controlled by appropriate gates at the inlet.

FLUMES

68. When, in order to avoid a long detour, it becomes necessary to carry the water of a canal across a valley, a **flume,** either of wood or of metal, is generally used. In the case of a city water supply, where all the installations must necessarily be on a much more permanent basis, these flumes would be replaced by masonry aqueducts, or they would, at least, be built of metal in a very substantial and perfect manner.

TIMBER FLUMES

69. General Description.—Timber flumes may be built in many ways. A simple form for a small flume, suitable for a cross-section of 4 ft. × 2 ft., is shown in Fig. 11. The dimensions of the timber are given in the figure. The bents may be 4 to 6 feet apart. In this very simple form of construction, no mortising need be used, as all the pieces can be assembled with spikes, bolts, and nails.

Fig. 11

An example of a larger and more perfect structure is shown in Fig. 12, which is a cross-section of the San Diego flume

in California. The inside width of this flume is 5 feet 10 inches, and the height, 3 feet 10 inches. On a hillside excavation, the flume rests on a bench 12 feet in width; while in crossing drainage lines, it rests on trestles. It is supported on mud-sills 12 in. × 2 in. laid crosswise of the bench and 4 feet apart. Longitudinal stringers of 4 in. × 6 in. timbers rest on the mud-sills, and on these are placed floor-

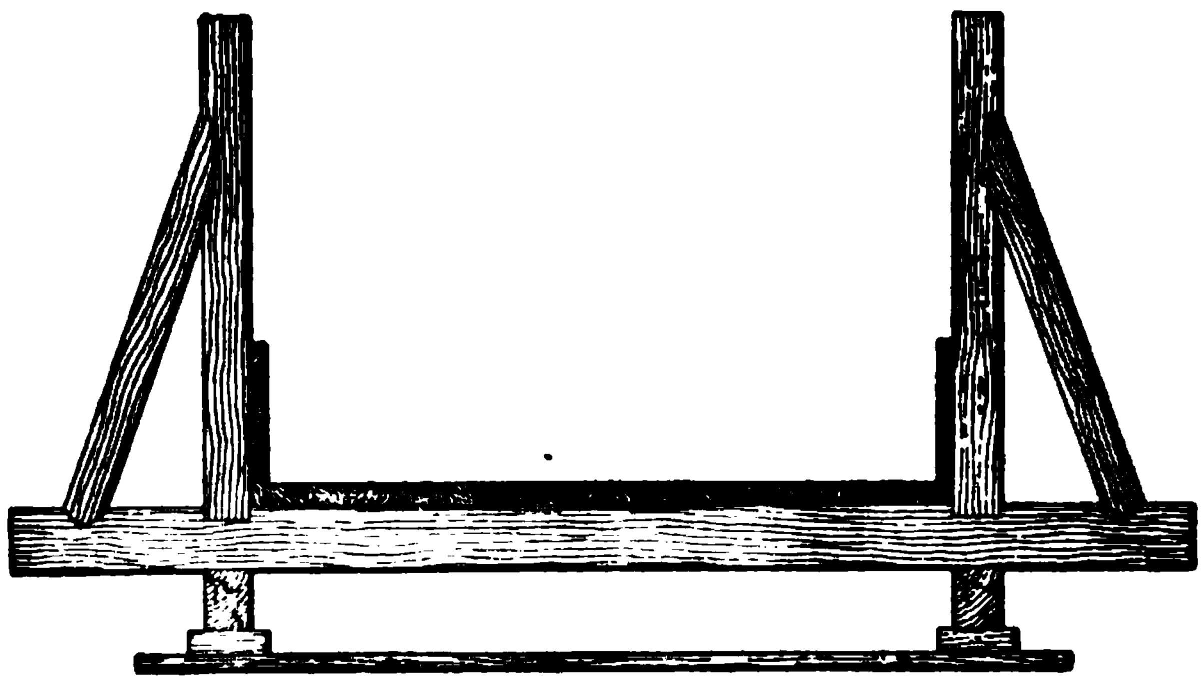


FIG. 12

beams 4 in. × 6 in., 4 feet apart. Uprights, 4 feet high, consisting of 4 in. × 4 in. scantling for supporting the sides, are let into the floorbeams, and are braced by short scantling let into the floorbeams and the posts. The interior is planked with 2-inch boards running longitudinally. Wire nails are best for spiking the planks to floorbeams, but the sides should be bolted at the joints. Nails will rot the side planking, as they are exposed alternately to air and water.

70. In the construction of wooden flumes, well-seasoned stuff should be used, and much better results, as regards flow and tightness, are obtained by having the edges and inside faces planed. A heavy coat of paint applied to the edges of well-matched planking just before spiking will make the box water-tight; without this, the joints must be calked with oakum. It is well to paint the whole of the inside, or at least the joints. When a flume is connected with an earthen canal, the greatest care must be taken to secure the point where the two connect against washing out. The flume

should enter well into the canal bank, and every possible outlet for the water should be stopped in the most effective manner.

71. Formula for the Flow of Water in Wooden Flumes.—The symbols having the same meanings as in the formulas for canals, the following formulas apply to wooden flumes:

$$v = \sqrt{\frac{100{,}000\, r^2 s}{6.6\, r + .46}} \qquad (1)$$

$$s = \frac{6.6\, r + .46}{100{,}000}\left(\frac{v}{r}\right)^2 \qquad (2)$$

Timber flumes are generally rectangular in shape, and may be very accurately proportioned to secure the best results. The most favorable rectangular cross-section is that in which the water has a depth equal to half the width.

EXAMPLE.—A timber flume, 10 feet wide and running 5 feet deep, has an inclination of 9 inches to the mile. What is its discharge in cubic feet per second?

SOLUTION.—Here $a = 10 \times 5 = 50$, and $p = 10 + 5 + 5 = 20$; whence, $r = \frac{50}{20} = 2.5$; also, $s = \frac{.75}{5{,}280} = .000142$. Substituting in the formula,

$$v = \sqrt{\frac{100{,}000 \times 2.5^2 \times .000142}{6.6 \times 2.5 + .46}} = 2.29 \text{ ft. per sec.}$$

The discharge is found by the formula $Q = a\,v$. In this case, $Q = 50 \times 2.29 = 114.50$ cu. ft. per sec. Ans.

72. It will often be required to find the dimensions of a flume to carry a given quantity of water under fixed conditions. The exact solution of this problem is difficult, since it leads to the solution of an equation of the sixth degree; by means of a system of trial and error, however, an approximate solution may easily be obtained that will give values within the practical limits required. The following illustrative example will make the method of operation clear:

It is required to compute the dimensions of a wooden flume to convey 250 cubic feet of water per second with a grade of $8\frac{1}{2}$ feet per mile, the width of the flume to be twice the depth of the water flowing through it.

Let x equal the depth of the water in the flume; then the width will be $2x$; the wetted perimeter, $4x$; the area of the water cross-section, $2x^2$; and the hydraulic radius, $2x^2 \div 4x = \frac{1}{2}x$.

The slope is $8.5 \div 5{,}280 = .0016$; and, since the discharge is to be 250 cubic feet per second, the mean velocity v must be $250 \div 2x^2 = \frac{125}{x^2}$.

Substituting the above terms in formula **1**, Art. **71**,

$$\frac{125}{x^2} = \sqrt{\frac{100{,}000 \times \frac{x^2}{4} \times .0016}{6.6 \times \frac{x}{2} + .46}}$$

Squaring, transposing, and reducing,

$$x^6 - 1{,}289x = 179.7 \qquad (1)$$

This equation is to be solved by trial. Assuming a depth of water of 5 feet for x, and substituting, we have, for the value of the left-hand member of equation (1), $5^6 - 1{,}289 \times 5 = 15{,}625 - 6{,}445 = 9{,}180$, which is much greater than the second member of the equation, and shows that the assumed value is too great.

Trying a value of $x = 4$, we have $4^6 - 1{,}289 \times 4 = 4{,}096 - 5{,}156 = -1{,}060$, which is less than the second member of equation (1), but nearer to it than the value obtained when 5 was substituted.

Trying 4.2, we have $4.2^6 - 1{,}289 \times 4.2 = 5{,}489 - 5{,}413.8 = 75.2$, which is still less than the required quantity. The value 4.3 gives $4.3^6 - 1{,}289 \times 4.3 = 6{,}321.5 - 5{,}542.7 = 778.8$, which is too great. It thus appears that a depth of water of 4.25 feet will satisfy the required condition very nearly, giving a width of flume of 8.5 feet.

IRON AND CONCRETE FLUMES

73. In the more modern works built abroad and in those built in the West by the Reclamation Service of the United States Geological Survey, steel and reinforced concrete are rapidly displacing wood as a material for the construction of

flumes. In the use of steel for long flumes, the expansion and contraction of the metal has introduced a difficulty of construction, though ordinarily of not sufficient moment to create serious trouble, since the steel usually possesses the same temperature as the water flowing through it. A steel flume is usually constructed after the pattern of a plate-girder bridge, of plate girders strengthened vertically by angles, and braced together at the top. The floor should be supported on floorbeams spaced about 5 feet apart.

74. With the introduction of reinforced concrete, structures of that material are being rapidly adopted wherever

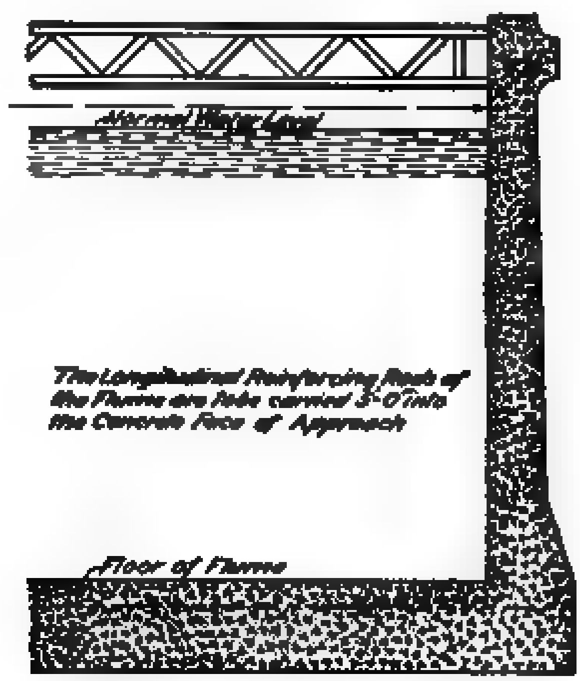


FIG. 13

substantial flumes are needed. Fig. 13 shows a large flume of this kind, used on the Minidoka canal, built by the American Government in Idaho. The width of the flume is 34 feet, and the depth of water inside, 10 feet, the extreme inside depth of the aqueduct being 12 feet. The floor is of concrete, 24 inches in thickness, braced with 1-inch steel rods spaced 6 inches between centers and turned up 3 feet into the upright sides at each end. The sides taper from 14½ inches near the floor to about 11 inches at the top, and are strengthened with two rows of ¾-inch steel rods, spaced about 9 inches on centers, running longitudinally through

the sides, and a row of 1-inch steel rods running vertically and spaced 6 inches on centers. The sides of the flume are tied together at the top by a low latticed girder.

BRIDGES AND TRESTLES FOR FLUMES

75. Bridges.—When the opening to be spanned by a flume is of considerable width, it will be necessary to use some form of trussed structure. Timber stringers should not be used for spans of more than 12 or 14 feet without trussing, unless the loads to be carried are light or the expense of trussing is much greater in proportion than the cost of the extra sizes of timber that would be required for the longer spans. In such cases, stringers with spans of 16 and even 20 feet are sometimes used. It is generally very difficult to get good, sound timber for stringers in sizes greater than 12 in. × 16 in. and 32 feet long, and even these dimensions are seldom used, owing to the expense, the difficulty of transporting such pieces, and the uncertainty in regard to their strength.

For spans between about 14 and 40 feet in length, combination king- or queen-post trusses can be conveniently used. For larger spans, plate girders, reinforced concrete structures, or any of the customary forms of trusses, as the Howe truss or the Pratt truss, may be employed.

76. Trestles.—For crossing very long depressions, trestles are as commonly employed in irrigation engineering as in railroad work. Indeed, they are more generally used in the former than in the latter, because in railroading they are frequently replaced with earthen embankments, which are to be preferred, as being more permanent for carrying trains; whereas it will rarely be found expedient to carry an irrigation flume or even a pipe line on an embankment, as the almost inevitable settling of the earth would seriously endanger the conduit. The subject of trestles is fully treated in *Trestles*, and here a few additional remarks will suffice.

When framed trestles are used, it is advisable to avoid inclined posts, mortising, and, as far as possible, different sizes of timber.

Figs. 14 and 15 represent the general features of a good system of trestling for moderate heights. The stuff used is all either 8 in. × 8 in. or 8 in. × 2 in. The 8 in. × 8 in. posts are set on the sills, which are also 8 in. × 8 in., either merely resting on the top face, or notched in ½ inch. They are held in place by **plaster plates**, of 8 in. × 2 in. stuff, bolted and spiked to posts and sills. Fig. 14 shows one of the posts and sills connected in this manner. The caps are connected with the upper ends of the posts in the same way. The posts are steadied by means of X bracing, of 2 in. × 8 in. stuff, as shown in the right-hand view in Fig. 15. The two pieces of the bracing are bolted together at the center, against an 8 in. × 8 in. block set between them; they are also bolted and spiked to posts, caps, and sills. These connections can be more perfectly made by first spiking the pieces together in place, and then putting in the bolts. In an emergency, all the connections may be made with spikes. The flume stringers are notched over the sills and are so disposed that joints will occur over the caps. These joints are secured by plaster plates bolted and spiked. The trestle is stiffened longitudinally by X bracing, of 2 in. × 8 in. stuff, as shown in the left-hand view in Fig. 15, butting under the flume stringers and against the sills, and secured laterally by plaster plates, chocks, mortising, or otherwise. The form of trestle here shown is suitable for moderate heights, say up

FIG. 14

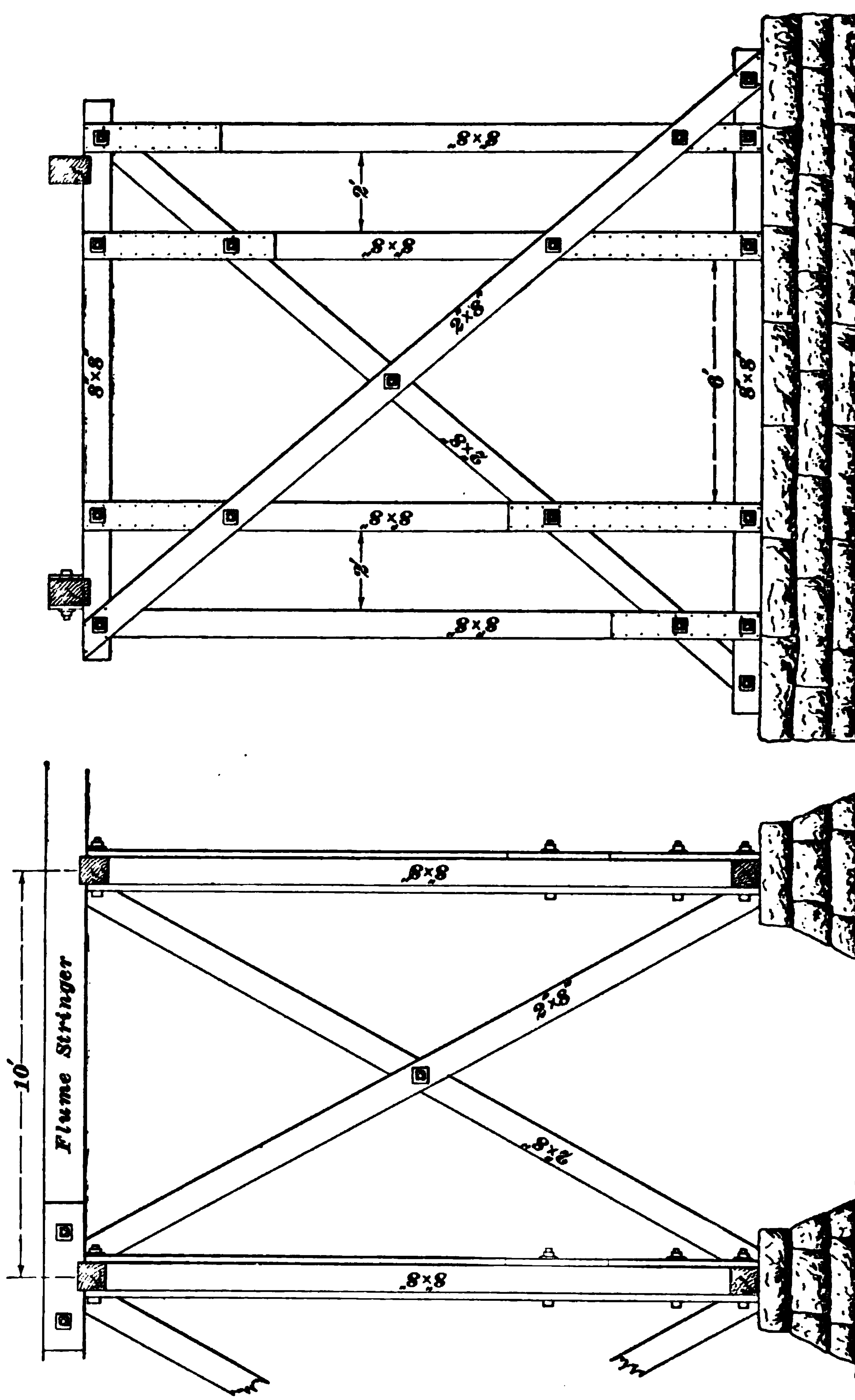


Fig. 15

to 20 feet; it will carry a 10 ft. × 15 ft. flume, and is about as light as will be perfectly satisfactory under this loading. If it were more convenient to use heavier stuff, the bents could be spaced farther apart.

PIPES AND TUNNELS

77. Pipes.—The best way to convey water for any purpose is by means of pipes. A pipe can be laid anywhere, subject to the conditions established in *Water Supply*, it is less subject to disaster and to loss of water by leakage, seepage, and evaporation, and the tendency of modern practice is to extend the use of pipes for the conveyance of water for irrigation, except where very large bodies are to be carried, when it will generally be impossible to avoid the use of canals having the dimensions of small rivers. The different kinds of pipes are fully treated in *Water Supply*.

78. Tunnels.—It frequently becomes necessary, in order to avoid long detours, to carry the line of conduit through a tunnel. If the conduit is an open canal, the tunnel will merely form an opening in the hill, affording a passage for the canal; if the conduit is a pipe line, running under pressure, the tunnel will generally form a continuation of the pipe, and will be entirely filled with water, running also under pressure. In either case, experience has abundantly shown that, except under very rare conditions, the tunnel should be lined throughout, even when it runs through rock. In modern tunneling, large quantities of high explosives are used, which greatly shatter the surrounding rock, so that fragments are continually coming away, particularly from the roof, when the tunnel is not lined, greatly interfering with the flow of water through it. Tunnels driven through earth must, of course, be thoroughly secured by lining. Such tunnels are frequently secured by timbering only, without, however, being always satisfactory. They have also, in the case of large conduits, been lined with wooden staves, as already described, the iron bands being replaced by concrete closely packed between the outside of the staves and the walls of the tunnel.

APPLICATION OF WATER TO THE GROUND

METHODS OF IRRIGATION

79. General Observations.—So far, only the collecting, storing, and transportation of water for irrigation have been considered. All these processes are merely preliminary to the great object of getting the water on the land for the purpose of producing crops; this, while the most important operation, being that up to which all the rest of the work has led, is in many respects the most complex. The problem is: Given a certain area of land, and a certain volume of water with which to irrigate it, how shall this water be evenly spread over the ground, so that every portion may receive a sufficient but not excessive degree of moisture, and that no water shall be wasted? There are many methods of applying the water; some of the principal ones will now be described.

80. Preparation of Soil.—It is essential so to prepare the soil that it will properly absorb the water applied by irrigation. The surface slope should be so fixed by grading, and the soil be put into such a condition of porosity by tilling, as to cause the water to flow over it slowly, and thus be easily absorbed. The subsoil may be opened by deep plowing, after which a thorough harrowing is necessary to place the soil in a proper condition of tilth. Successful irrigation is dependent on careful cultivation, which implies constant attention to a small area of land by each individual cultivator. Better crops may be obtained from the careful cultivation of 10 or 15 acres by a single farmer than by the careless or improper cultivation of several times that area.

81. Irrigation by Sprinkling.—Sprinkling is a method with which, when practiced on a small scale, all

are familiar. When a flower bed is to be watered, it is sprinkled by means of a watering pot. If a larger space is to be operated on, a hose with perforated nozzle, or "rose," is used, or a watering cart may be employed, delivering water in the form of a spray.

There can be no doubt that sprinkling is the best way in which water can be applied to the soil. This method fulfils all the requirements of uniform distribution, moderate and easily regulated amount, and, in consequence, permits a gradual absorption of the moisture without supersaturation of the soil or the presence of exposed surfaces of unabsorbed water, which must pass off by evaporation, thus adding to the ill effects of alkali already mentioned. The method can also be applied to rough and uneven land, without the necessity of any previous grading.

The objection to this method is the difficulty of applying it on the very large scale that is sometimes needed. It is probable, however, that by a careful and judicious system of piping it could be made more available than it is at present. It is already largely used in Florida, and is thus described by George W. Adams, of Thonotosassa: "I have a 25-horsepower horizontal boiler and a 12 in. × 7 in. × 10 in. duplex pump, with 6-inch main pipe and 3-inch laterals at the main, and running down to one and a half at extreme ends. My trees are 21 feet apart each way. I have a hydrant in the center of every 16 trees. I use the McGowan automatic sprinklers, connecting the sprinkler with hydrants by a 1-inch wire-wound rubber hose 50 feet long. I use twelve of the sprinklers at one time, and could use more just as well, each sprinkler staying in place 30 minutes, each one covering a space of from 50 to 70 feet, according to the amount of pressure given them, and discharging about 1,000 gallons. By this process I have a genuine rain, either a light one or a powerful one, at pleasure. If I wish to throw water over the tops of the trees, I use the nozzle instead of the sprinkler. I run the pump from 7 A. M. to 6 P. M. without stopping, using less than ½ cord of wood in 11 hours. I find no bad results from applying the water in the hottest sunshine, but

would if I applied it through an open hose. I think the sprinkler method of applying water requires less help than any other I have seen, and is without any danger to fruit or tree. The fireman can manage the sprinklers within reasonable distance of the pumping station. For other portions, only one man is ever needed, and it is light work for him."

In this method, it will generally be necessary to use a pump, either directly forcing the water through a hose, or else raising it into an elevated tank, from which it may be drawn as wanted.

82. **Irrigation by Flooding.**—Next to sprinkling, flooding is the method of applying water to the land that most naturally suggests itself. It consists essentially in spreading the water in a thin sheet over the area to be irrigated, and this may be accomplished in several ways. The first feature of this method is a ditch or canal running along the upper border or highest level of the field to be irrigated. This may be either the main conduit itself or a subsidiary ditch fed from it. It will run, with a slight fall, following the highest level, or contour line. When it is desired to irrigate the land from this ditch, one of two ways will be adopted. Small temporary obstructions, such as a few shovelfuls of earth, may be placed in the ditch, causing it to overflow at the desired points in a thin sheet. Or a break may be made in the ditch, allowing water to escape and spread over a certain portion of the field, which break will then be closed, and a new one opened, a little in advance of the first, which will irrigate an adjacent portion, and so on, opening and closing breaks with the shovel, until water has been spread with more or less regularity over the whole field. If a flume or pipe line is used instead of a ditch, sluices, permanent sliding gates, or hydrants must be placed at convenient points.

This system presupposes that the land lies on a gentle and regular slope, such as characterizes many portions of the Western American states. Almost invariably, however,

some preparatory work must be done in leveling and grading the land before this method can be employed.

The flooding system of irrigation is largely used in the cultivation of alfalfa and grass crops. It is the simplest and cheapest method, but it is very wasteful of water, and fulfils very imperfectly the requirements of a satisfactory irrigation. It distributes the water with great irregularity, overwatering some portions and scarcely moistening others. Its imperfections may be to some extent neutralized by carefully watching and guiding the progress of the water as it comes from the ditch, causing it to deviate here and there from its natural course, by a judicious use of the hoe and shovel.

There are several modifications of the general system, better adapting it to the varying topography of the ground. When a comparatively steep hillside is treated in this way, it will be better to run a series of ditches across the slope, dividing it into belts or zones, than to depend on a single ditch to irrigate the whole slope. In this case, the unabsorbed water that flows over the first belt will be caught in the ditch next below it, and passed on to the next belt, and so on.

If the ground is very level, the difficulty that then presents itself is the too rapid absorption of water in the vicinity of the outlet of the ditch before it can reach the farther limit of the field. In this case, the check-system is a useful modification.

83. In the **check-system**, a series of small ridges or checks are run across the slope, parallel or nearly so to the ditch, dividing the ground into a series of belts, in the same way as is done by the subsidiary ditches just described for hillside work. The first belt is flooded, and the water allowed to stand on it until the ground has become sufficiently saturated. The water is then drawn off by means of a break made in the ridge, and allowed to flood the next zone below, and so on.

84. Checker-Board System.—The **checker-board system** is a modification of the check-system, and is especially suited to very level land. It consists in crossing the

checks with others, more or less nearly at right angles to them, by which the whole territory to be flooded is divided into compartments. These are flooded, one or more at a time, and the water that is not absorbed is passed on to adjacent compartments.

85. The **method by furrows** is considered a very excellent one. Instead of flooding the surface generally, as just described, small ditches, or furrows, are run from the main feeder, at such an angle across the slope as to insure a gradual descent, and the water is allowed to enter these and flow through tnem over the field to be irrigated, the water being distributed by lateral absorption and percolation instead of by flooding the surface. As in all other systems, the course of the water should be watched and guided as far as possible, and irrigation should be followed by cultivation, or stirring and working the soil.

When orchards are irrigated in this manner, the furrows are run among and around the trees. If the land is laid out before the trees are set out, the furrows are established first, according to the most advantageous manner of suiting them to the topography of the field, and the trees are planted to suit the position of the furrows.

The ends of the furrows are connected, so that water may circulate in all directions. The water is guided by opening or closing the furrows with the shovel. Obviously, the ground must be tolerably regular in order to permit the use of this method.

86. Subsoil Irrigation.—This system consists in running a series of pipes, generally from 1 foot to 18 inches below the surface, very much after the manner of drain pipes. Water may be admitted into these pipes at the upper end, the lower end being closed temporarily, and allowed to escape either through perforations in the pipe or through their loose joints, if common drain pipe is used. After proper saturation, the lower ends of the pipes are opened, and they then form a drainage system for the removal of superfluous or non-absorbed water; or, the joints of the pipes

may be made tight, with openings and vertical pipes at certain intervals, from which water may flow and spread over the ground in the vicinity. This, however, constitutes rather a modification of the flooding system.

Although certain practical difficulties have been encountered in the application of this method, it gives promise of being one of the best that can be devised. It must necessarily be that which is most economical of water, and when used as first described, in the manner of drain pipes, allows the water to be drawn up by capillary action instead of sinking down by the action of gravity; loss by evaporation is thus reduced to a minimum.

87. Other Methods of Irrigating.—While there are many other systems practiced, they will be found, on examination, to consist of modifications of those described, which may be considered, therefore, as typical. The very important subject of sewage irrigation is treated in *Sewage Purification and Disposal.*

GAUGING WATER FOR USERS

88. Introductory.—When an individual owns a plant for watering his own lands, it is not necessary that the amount of water used should be measured. When the water flowing in a canal is furnished by a company or the state to more than one irrigator, some system must be adopted for measuring the amount furnished. Charges for water rates are usually based on the number of cubic feet per second delivered, and the user should know whether or not he is getting the amount of water for which he contracts.

89. Rough Methods of Gauging.—Persons whose duty it is to divide the water on behalf of a company have, as a rule, a limited knowledge of hydraulics and are capable of using but simple instruments. A very common practice on canals owned by companies is for the gate tender to judge the amount of water by its appearance. Absolute quantities are often not furnished but only proportional parts of the flow. Thus, an irrigator receives a tenth or a fifth of the

water in the ditch, rather than the number of cubic feet per second contracted for.

At the point of diversion, a wooden flume, or, in substantial works, a concrete lining, is placed in the canal, with a gate in the main canal that can be raised or lowered so as to permit or retard the passage of a portion of the water. Just above it in the side of this flume is the head admitting the water into the farmer's lateral, and this is closed by a gate consisting usually of a board that may be raised or lowered so as to admit the proper amount. Often, where water is scarce, persons in charge of the regulation of these gates, after setting them to the proper amount of opening, lock them with padlocks to prevent their being tampered with by the water users.

90. Weir Measurements.—Where greater accuracy of water distribution is desired, weirs are used. Tables showing the discharge of the weirs for certain lengths of crest and depth over the crest are furnished the men in charge of the distribution, so that they may know exactly how much water is being delivered in this manner. The practice of constructing measuring weirs in the heads of distributaries and of rating the discharge of these at various depths is rapidly being introduced into the arid regions of the United States, and the trapezoidal, or Cippoletti, measuring weir is growing in favor. The laws of Colorado and of some other states require the construction of measuring weirs, and water commissioners are employed whose business it is to ascertain the amount flowing through the various heads. The results are tabulated and furnished to the canal owners for their information.

91. Foote's Water Meter.—Several contrivances, more or less ingenious and efficient, have been devised for the purpose of readily determining the amount of water furnished to consumers. These devices are sometimes called **modules**. In order that they may meet all requirements, they should not only measure the water furnished, but also deliver a certain specified quantity.

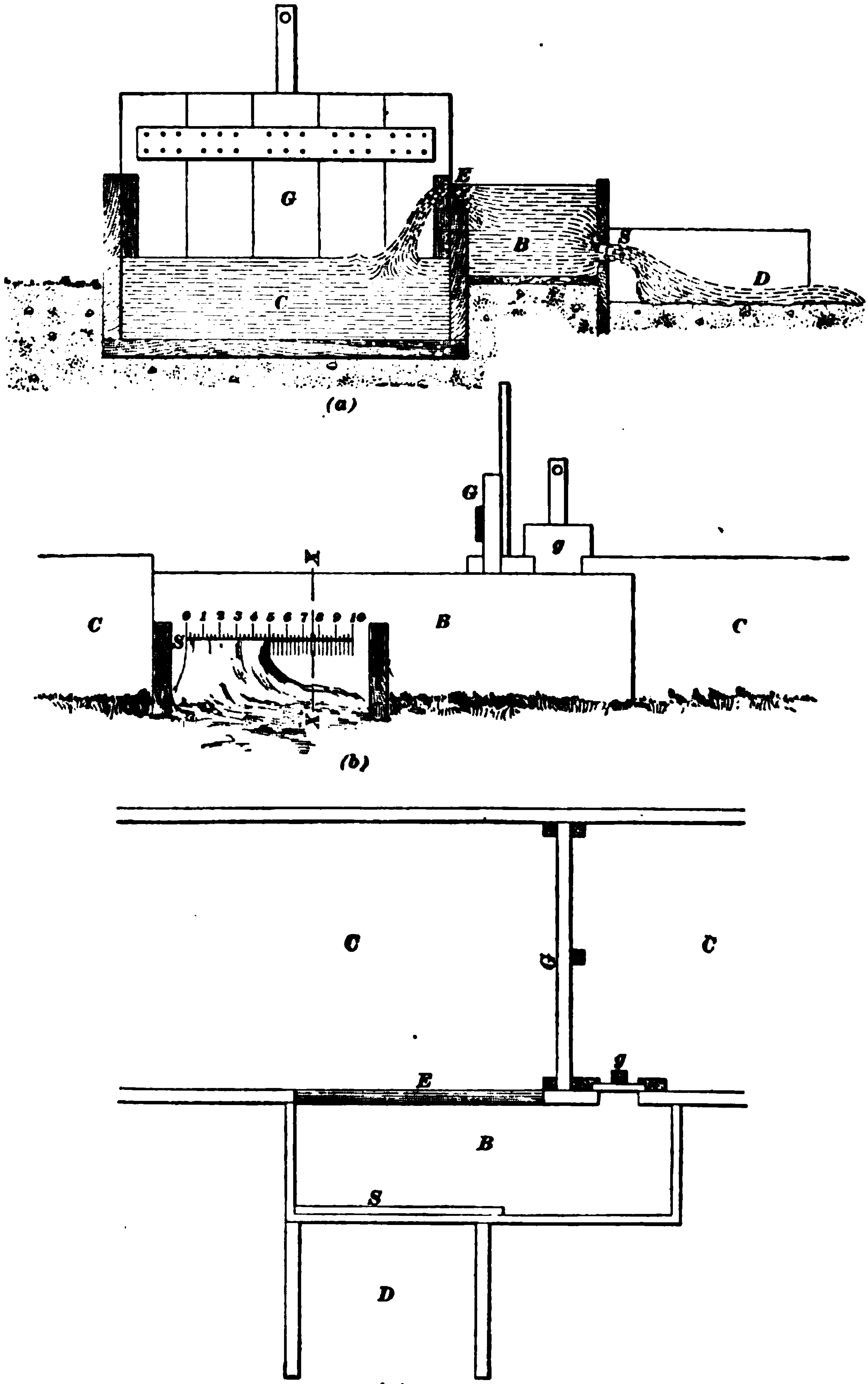


FIG. 16

Among the appliances proposed for the delivery of a specified quantity of water in a given time, one of the best is that invented by A. D. Foote, of Idaho. The general features of this meter are shown in Fig. 16.

In Fig. 16 (*a*) is represented a cross-section through the main canal or flume *C*, from which it is desired to draw a certain measured volume of water to feed the ditch *D* by means of a slot *S* in the box *B*. In order to effect this, the sliding gate *G* is partly closed, so as to impede the free flow of water in the canal *C*, and force a portion of it into the box *B*, through the small sliding gate *g*, which is partly opened for the purpose. After a few trials with the two sliding gates, their openings will be so adjusted that the proper level of the water in the box is maintained, the surplus passing over the edge *E*, and falling back again into the canal *C*, below the gate *G*.

The slot *S* is opened more or less by means of a slide on the inside of the box, the width of opening being recorded by a scale shown in Fig. 16 (*b*). This contrivance, as designed by Mr. Foote, gives the amount of water delivered in miner's inches, but the scale can be laid out so as to give the quantity in any other unit.

CROPS

92. The preceding articles cover the strictly engineering and commercial features of irrigation—all those, that is, which relate to securing the water and turning it over to the farmer, on profitable terms, for utilization. It is necessary, however, that the well-equipped engineer in this specialty should have some knowledge of how crops are raised and how the water is handled in the process.

The successful raising of crops by the aid of artificial irrigation is a scientific operation, the principles of which must be carefully studied in order to insure the best results. Long experience shows that crops do best when they receive the minimum quantity of water that they require, and, it may be added, the maximum amount of cultivation. It is found,

also, in bringing in new land, that more water is required the first year than in subsequent ones. It appears that, by a free application of water at the outset, the soil becomes gradually saturated; so that year by year the amount of water necessary for plant life diminishes, until it reaches a constant amount very much less than the original quantity required.

No fixed rules can be laid down regarding the exact amount of water required for each crop; experience, observation, and judgment are necessary to success. Some generalities, however, will be useful.

93. Alfalfa.—"Alfalfa is the greatest forage plant the world has ever known, and it should be a special crop with every irrigation farmer. It is known scientifically as *medicago sativa*, its botanical name. In the Spanish language it is called "alfalfa," while the French, Swiss, German, and Canadian people call it "lucerne." It is a leguminous perennial, and properly belongs to the pea-vine family. It is often miscalled a grass. Its term of existence has not been authentically established, but it will last the average age of a man, and instead of depleting the soil, it has a way, through its root nodules, of constantly replenishing the soil with the nitrogenous fertilizing elements of the atmosphere."—*Wilcox.*

A porous subsoil, which promotes drainage and prevents unabsorbed water from standing on the surface, is advantageous for this crop, as indeed for all others. Thorough plowing should be done in the fall, and the ground leveled before seeding in the spring. A good flooding is needed just before seeding. The seed is covered by light harrowing, or planted with a drill. It should not be buried more than 1 inch to 1½ inches deep. Too early irrigation after seeding should be avoided; it should not be practiced, as a general rule, until the plants are nearly or quite 1 foot high. When the plant has taken full possession of the ground, one good irrigation after each cutting will usually be sufficient.

"Plowing under green alfalfa as a manurial agent and soil restorative is becoming recognized in the West as a

very essential agency in preventing soil deterioration. It is, therefore, a very useful plant in following out a line of crop rotation. As a green manure or soil renovator, alfalfa is hardly equaled by any other plant. It is very rich in phosphoric acid, potash and lime, and gets a goodly portion of nitrogen from the air, leaving much of this in the soil by means of its large roots. Aside from this, when used as a green manure, there is a great deal of humus added to the soil, both by the matter turned under and by the roots. The large, long roots open the subsoil to a great depth, serving much the same purpose as the subsoil plow."

94. Wheat.—Wheat requires high land. The ground should be moist before seeding. Harrow, and plant with press drill. First irrigation may take place when the plant is 5 or 6 inches high. A second lighter irrigation may be needed about a month after the first. A third irrigation is sometimes given just as the grain is heading, if the ground has not kept sufficiently moist.

95. Oats.—Oats are treated much as wheat except that they require considerably more water. The heaviest irrigation—sometimes amounting to 1 foot in depth—is given when the plant is about 6 inches high.

96. Rye.—Rye is the easiest grown of all the cereals, and needs the least water; sometimes only one light watering is sufficient.

97. Corn.—Corn requires a great deal of preparation of the soil, and of cultivation after planting. Excessive irrigation must be avoided. One or two irrigations will be sufficient. A watering will generally be wanted when the tassels are formed. Altogether, this crop requires a good deal of attention.

98. Grasses.—Much that has been said of alfalfa applies to the grass crops. One general rule for hay crops is not to irrigate for a considerable time previous to cutting, so as to permit a thorough assimilation of plant food, and to allow the ground to acquire a proper condition for cutting and curing the hay.

AMERICAN LAWS ON IRRIGATION

99. Priority of Appropriation, and Prorating.—The questions of riparian rights and the right of priority of ownership of water have assumed a new aspect under changed conditions. This has not yet taken definite shape, and there is no set code regulating the rights of water for use in irrigation. The old law of **riparian rights** can be enforced only for protection of water that has been put to a beneficial use. The right of **priority of appropriation** was maintained for a long time in some of the states of the West, but that also has practically been superseded by the principle of **beneficial use.** Instead of the ownership of the water going with the land, as under the older common law, the flowing water is not classed as property that can be owned by any person. Where the land originally belonged to the United States, the unused waters both above and beneath the land still belong to the government. The use of the water is guaranteed to appropriators to the extent to which they put it to beneficial use, and generally in the order of priority in which they make such use. While the right of appropriation was originally based on the need of water and the right to take it, changed conditions have resulted in its modification by requiring evidences of beneficial use.

A still later development has been the system known as **prorating.** This consists in dividing water proportionally to the amount available. No consideration is given the user nearest the ditch head, who may have been the first irrigator. He receives the same proportion of his usual share as do his associates. In time of scarcity of water, the application of priorities must give way to proportional division of the supply available. It is not possible to deprive a large number of the water users of their supply and cause their crops to be destroyed, in order that the full proportion may be given the favored few prior owners. Accordingly, there is a tendency

to abandon the strict observance of priorities in favor of prorating water. The practice of judicial decision seems to be in favor of the view of changing the use of the water to which an irrigator is entitled from one piece of land to another, but this right could only be held to be pertinent to some specified piece of land, and not to land in general, or wherever the irrigator might desire to use it. In other words, his right to water would not be owned separate and apart from the land.

These same questions become still more complicated when considered in connection with property in water owned by a canal company. It is then necessary to recognize the difference between rights to divert water, to carry it, and to furnish water to users and to charge for it. Such rights are distinct from those bearing on the actual use of water in irrigating the land, and are considered to be enjoyed by a canal company as a common carrier. The company has no actual ownership in the water in the sense that it owns the canal and the regulating works; its position is that of a trustee conveying the water to those who will put it to actual beneficial use.

100. Reclamation Law.—The President of the United States approved June 17, 1902, an Act known as the **Reclamation Law.** This created a means whereby the proceeds of the sales of public lands in thirteen arid states will become a revolving fund held in the Treasury of the United States, and known as the **Reclamation Fund,** to be used in the examination, survey, construction, and maintenance of irrigation works for the storage, diversion, and development of waters for the reclamation of the lands in those states and territories. The law further provided that the reclaimed lands should ultimately be sold to bona-fide settlers at a uniform price per acre that would be sufficient to reimburse the government for the entire outlay, such proceeds becoming again available for further reclamation. In accordance with this law, and to carry out its provisions, a branch of the United States Geological Survey, known as the Reclamation

Service, was established. The Service examines the lands, determines their possible capability of reclamation, recommends their withdrawal from sale or preemption under other laws, prepares plans for projects for their reclamation, and, if the latter are approved by the Secretary of the Interior, undertakes the construction, and, if necessary, the subsequent maintenance of the works.

HYDRAULIC TABLE FOR LONG PIPES

Although problems relating to the flow of water in long pipes can be solved by formulas, these formulas are often cumbersome and their application requires much labor. As, from the nature of the subject, exact results cannot be obtained (errors of from 2 to 10 per cent. are not considered unusual, and may be expected), the work is much facilitated by the use of tables. The one here given (found in print nowhere outside of the I. C. S. publications), is very conveniently arranged, and will save considerable time and labor. It comprises every commercial size of cast-iron water pipe, and is equally applicable to wrought-iron and steel pipe, not riveted.

It must be borne in mind that the projecting rivet heads in a steel riveted pipe reduce its carrying capacity very much more than by the decrease of the diameter caused by the annular ring of rivets. For instance, a 42-inch riveted steel pipe with a row of rivet heads projecting into the interior, 1 inch in depth, will not have the same discharge as a 40-inch smooth pipe. Costly and embarrassing mistakes have been committed by neglecting this fact.

The quantities given in the table are:

d = diameter, both in inches and in feet (the value used in the formulas is always in feet, unless otherwise stated).

v = velocity of flow, in feet per second.

$s = \frac{h}{l}$ = slope, or head per unit of length of pipe, or sine of the average inclination of the pipe to the horizontal (here, h is the total head, and l is the length of the pipe).

$s_m = 5{,}280 \frac{h}{l}$ = head, in feet, per mile of pipe.

$G = \frac{l}{h}$ = grade = length of pipe for which the head, or rise, is 1. If the unit used is the foot, the quantity G

indicates the number of feet of pipe in which the rise is 1 foot. Thus, if $G = 750$, the grade is 1 foot in 750 feet.

Q = discharge, for clean or tar-coated pipes, in either cubic feet per second, gallons per minute, or gallons per day of 24 hours.

$Q' = \frac{Q}{\sqrt{2}}$, corresponding quantities for extremely foul pipes.

The head h and length l are easily found when either the slope or the grade is given, since $h = s\,l$, and $l = G\,h$.

The table has been constructed from the formulas

$$v = \sqrt{\frac{2ghd}{fl}} = \sqrt{\frac{2gd}{f} \times \frac{h}{l}} \qquad (1)$$

$$Q(\text{cu. ft.}) = \frac{\pi d^2}{4} v \qquad (2)$$

In these formulas, f is an empirical coefficient that varies with v and d. The values of f have been taken from special tables. It has been assumed that, for extremely rough or foul pipes, the value of f is twice that for clean pipes. If the velocity in a rough pipe is denoted by v', and $2f$ is used instead of f, equation (1) becomes

$$v' = \sqrt{\frac{2gd}{2f} \times \frac{h}{l}}$$

Therefore, $$\frac{v}{v'} = \frac{\sqrt{\frac{2gd}{f} \times \frac{h}{l}}}{\sqrt{\frac{2gd}{2f} \times \frac{h}{l}}} = \sqrt{2}$$

and, as the discharges are proportional to the velocities,

$$\frac{Q}{Q'} = \frac{v}{v'} = \sqrt{2}; \text{ whence } Q' = \frac{Q}{\sqrt{2}}$$

In determining the diameter of a pipe, it is always advisable to determine it for both of the extreme conditions; that is, both assuming the pipe perfectly clean and assuming it extremely foul or rough. Also, when the diameter of a pipe is known, the values of Q and Q' show the extreme limits between which the discharge may vary.

EXAMPLE 1.—What is the discharge, in cubic feet per second, of a 14-inch pipe in which the velocity is 3.2 feet per second?

SOLUTION.—Find, in the table, under diameter 14 inches, 3.2 in the column headed v. Opposite this value and in column headed Cubic Feet per Second, the discharge is found to be 3.4208 cu. ft. per sec. Ans.

EXAMPLE 2.—Determine the velocity, in feet per second, in a 16-inch water main 1,500 feet long, with a head of 54 feet.

SOLUTION.—The ratio $\frac{l}{h}$ is $\frac{1,500}{54}$, or 27.778. Looking in the table, in the column headed G, under the diameter 16 in., it is seen that the value 27.778 falls between that corresponding to a velocity of 12.5 ft. per sec. and that corresponding to a velocity of 13 ft. per sec. The difference in the value of v for a difference in G of $28.782 - 26.681 = 2.101$ is $13.0 - 12.5 = .5$ ft. per sec. For a difference in G of $28.782 - 27.778 = 1.004$, the difference in v is

$$\frac{.5 \times 1.004}{2.101} = .24$$

Therefore, the velocity is $12.5 + .24 = 12.74$ ft. per sec. Ans.

EXAMPLE 3.—Required the diameter of pipe necessary to deliver 700,000 gallons per day of 24 hours, if the reservoir is situated 90 feet above the city and at a distance of 15,300 feet.

SOLUTION.—Here, $\frac{l}{h} = \frac{15,300}{90} = 170$. Looking for the number 700,000 in the column headed Gallons per Day, under diameter 6 inches, it is seen that the next higher, 761,360, requires a grade (column G) of 1 ft. in 38.374 ft. Since the available grade is only 1 ft. in 170 ft., look in the same column under the diameter 8 in. The next higher value is 721,890, and the required grade (column G) is 1 ft. in 175.36 ft. Therefore, an 8-in. pipe can be used, although the discharge will be somewhat greater than 721,890 gal. per day. Ans.

EXAMPLE 4.—It is required to deliver 2,500,000 gallons per day of 24 hours with a 20-inch pipe. If the reservoir is 9 miles from the city: (*a*) what must be the head? (*b*) what is the velocity in the pipe?

SOLUTION.—(*a*) Looking in the table, under diameter 20 in., in the column headed Gallons per Day, the value 2,537,900 is found. Opposite this value, in the column headed s_m, the quantity 3.4086 is found. This is the head per mile of length; therefore, the required head is $3.4086 \times 9 = 30.6774$ ft. Ans.

(*b*) Opposite 2,537,900, in the column headed v, the value 1.8 is found. Therefore, $v = 1.8$ ft. per sec. Ans.

HYDRAULIC TABLE FOR CAST-IRON PIPES

$d = 4$ inches $= \frac{1}{3}$ foot

v	$s = \frac{h}{l}$	$s_m = 5{,}280\,\frac{h}{l}$	$G = \frac{l}{h}$	Q: Cubic Feet per Second	Q: Gallons per Minute	Q: Gallons per Day	Q′: Cubic Feet per Second	Q′: Gallons per Minute	Q′: Gallons per Day
.1	.000014235	.075159	70,250	.0087265	3.9165	5,639.6	.0061704	2.7693	3,987.8
.2	.000056343	.29749	17,748	.017453	7.8329	11,279	.012341	5.5386	7,975.5
.3	.00012593	.6649	7,940.9	.026179	11.749	16,919	.018511	8.3079	11,963
.4	.00022149	1.1694	4,514.9	.034906	15.666	22,558	.024682	11.077	15,951
.5	.00034375	1.8150	2,909.1	.043632	19.582	28,198	.030852	13.847	19,939
.6	.00049163	2.5958	2,034.0	.052359	23.498	33,838	.037023	16.616	23,927
.7	.00066460	3.5091	1,504.7	.061086	27.415	39,477	.043194	19.385	27,915
.8	.00086208	4.5518	1,160.0	.069812	31.331	45,117	.049364	22.154	31,902
.9	.0010850	5.7288	921.66	.078538	35.248	50,756	.055534	24.924	35,890
1.0	.0013283	7.0136	752.82	.087265	39.164	56,396	.061704	27.693	39,878
1.1	.0015982	8.4386	625.69	.095990	43.080	62,036	.067875	30.462	43,865
1.2	.0018913	9.9860	528.74	.10472	46.997	67,675	.074045	33.232	47,853
1.3	.0022070	11.653	453.10	.11345	50.913	73,315	.080216	36.001	51,841
1.4	.0025487	13.457	392.36	.12217	54.830	78,955	.086387	38.770	55,829
1.5	.0029090	15.359	343.76	.13090	58.746	84,594	.092557	41.539	59,876

1.6	.0032955	17.400	303.44	.13962	62.663	90,234	.098728	44.309	63,804
1.7	.0037043	19.558	269.97	.14835	66.580	95,874	.10490	47.078	67,792
1.8	.0041346	21.830	241.86	.15708	70.495	101,510	.11107	49.847	71,779
1.9	.0045797	24.181	218.36	.16580	74.412	107,150	.11724	52.616	75,767
2.0	.0050596	26.715	197.64	.17453	78.328	112,790	.12341	55.386	79,755
2.1	.0055535	29.323	180.07	.18326	82.245	118,430	.12958	58.155	83,743
2.2	.0060679	32.038	164.80	.19198	86.160	124,070	.13575	60.924	87,730
2.3	.0065928	34.810	151.68	.20071	90.078	129,710	.14192	63.694	91,719
2.4	.0071461	37.731	139.94	.20944	93.994	135,350	.14809	66.463	95,706
2.5	.0077192	40.757	129.55	.21816	97.910	140,990	.15426	69.232	99,694
2.6	.0083111	43.882	120.32	.22689	101.83	146,630	.16043	72.001	103,680
2.7	.0089217	47.106	112.09	.23561	105.74	152,270	.16660	74.770	107,670
2.8	.0095660	50.508	104.54	.24434	109.66	157,910	.17278	77.541	111,660
2.9	.0102300	54.015	97.750	.25307	113.58	163,550	.17895	80.310	115,650
3.0	.010914	57.625	91.626	.26179	117.49	169,190	.18511	83.079	119,630
3.1	.011618	61.341	86.076	.27052	121.41	174,830	.19129	85.848	123,620
3.2	.012341	65.161	81.029	.27925	125.33	180,470	.19745	88.618	127,610
3.3	.013084	69.083	76.430	.28797	129.24	186,100	.20362	91.386	131,600
3.4	.013846	73.107	72.222	.29670	133.16	191,750	.20980	94.156	135,580
3.5	.014627	77.229	68.368	.30543	137.08	197,390	.21597	96.926	139,570

$d = 4$ *inches* $= \frac{1}{3}$ *foot*

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
3.6	.015426	81.448	64.826	.31415	140.99	203,030	.22214	99.694	143,560
3.7	.016244	85.767	61.562	.32288	144.91	208,670	.22831	102.46	147,550
3.8	.017106	90.321	58.457	.33160	148.82	214,310	.23448	105.23	151,530
3.9	.017962	94.837	55.674	.34033	152.74	219,940	.24065	108.00	155,520
4.0	.018836	99.451	53.091	.34906	156.66	225,580	.24682	110.77	159,510
4.1	.019750	104.28	50.634	.35778	160.57	231,220	.25299	113.54	163,500
4.2	.020684	109.21	48.346	.36651	164.49	236,860	.25916	116.31	167,490
4.3	.021629	114.20	46.234	.37524	168.41	242,500	.26533	119.08	171,470
4.4	.022601	119.33	44.246	.38396	172.32	248,140	.27150	121.85	175,460
4.5	.023593	124.57	42.386	.39269	176.24	253,780	.27767	124.62	179,450
4.6	.024604	129.91	40.643	.40142	180.16	259,420	.28384	127.39	183,440
4.7	.025634	135.35	39.010	.41015	184.07	265,060	.29001	130.16	187,430
4.8	.026672	140.83	37.493	.41887	187.99	270,700	.29618	132.93	191,410
4.9	.027739	146.46	36.050	.42760	191.91	276,340	.30236	135.70	195,410
5.0	.028824	152.19	34.693	.43632	195.82	281,980	.30852	138.47	199,390

5.5	.034510	182.21	28.977	.47995	215.40	310,180	.33938	152.31	219,330
6.0	.040634	214.55	24.610	.52359	234.99	338,380	.37023	166.16	239,270
6.5	.047293	249.71	21.145	.56722	254.57	366,570	.40108	180.00	259,200
7.0	.054394	287.20	18.384	.61086	274.15	394,780	.43194	193.85	279,150
7.5	.062021	327.47	16.124	.65449	293.73	422,970	.46279	207.70	299,080
8.0	.070089	370.07	14.268	.69812	313.31	451,170	.49364	221.54	319,020
8.5	.078518	414.57	12.736	.74175	332.90	479,370	.52449	235.39	338,960
9.0	.087345	461.18	11.449	.78538	352.48	507,560	.55534	249.24	358,900
9.5	.096814	511.18	10.329	.82901	372.06	535,760	.58619	263.08	378,830
10.0	.10672	563.46	9.3707	.87265	391.64	563,960	.61705	276.93	398,780
10.5	.11704	617.95	8.5444	.91628	411.23	592,160	.64790	290.78	418,720
11.0	.12777	674.62	7.8265	.95991	430.80	620,360	.67875	304.62	438,650
11.5	.13891	733.45	7.1988	1.0036	450.39	648,560	.70961	318.47	458,600
12.0	.15045	794.35	6.6469	1.0472	469.97	676,750	.74046	332.32	478,530
12.5	.16281	859.61	6.1423	1.0908	489.55	704,950	.77131	346.16	498,470
13.0	.175620	927.26	5.6942	1.1344	509.13	733,150	.80216	360.01	518,410
13.5	.18887	997.24	5.2946	1.1781	528.71	761,340	.83301	373.85	538,340
14.0	.20258	1,069.6	4.9363	1.2217	548.30	789,550	.86387	387.70	558,290
14.5	.21712	1,146.4	4.6059	1.2654	567.89	817,750	.89473	401.55	578,230
15.0	.23213	1,225.7	4.3078	1.3090	587.46	845,940	.92557	415.39	598,160

d = 6 inches = .5 foot

v	$s = \frac{h}{l}$	$s_m = 5\,280\frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
.1	.0000090795	.047939	110,140	.019635	8.8121	12,689	.013884	6.2310	8,972.6
.2	.000036020	.19018	27,763	.039270	17.624	25,379	.027768	12.462	17,945
.3	.000080595	.42554	12,408	.058905	26.436	38,068	.041651	18.693	26,918
.4	.00014189	.74917	7,047.7	.078540	35.248	50,758	.055535	24.924	35,891
.5	.00022015	1.1624	4,542.4	.098175	44.061	63,447	.069419	31.155	44,863
.6	.00031433	1.6596	3,181.4	.11781	52.873	76,136	.083303	37.386	53,836
.7	.00042479	2.2428	2,354.1	.13745	61.685	88,826	.097187	43.617	62,809
.8	.00055163	2.9126	1,812.8	.15708	70.497	101,520	.11107	49.848	71,781
.9	.00069312	3.6597	1,442.7	.17671	79.309	114,200	.12495	56.079	80,753
1.0	.00085075	4.4919	1,175.4	.19635	88.121	126,890	.13884	62.310	89,726
1.1	.0010219	5.3954	978.61	.21598	96.933	139,580	.15272	68.541	98,698
1.2	.0012071	6.3737	828.40	.23562	105.75	152,270	.16661	74.772	107,670
1.3	.0014083	7.4357	710.08	.25525	114.56	164,960	.18049	81.003	116,640
1.4	.0016236	8.5724	615.93	.27489	123.37	177,650	.19437	87.235	125,620
1.5	.0018526	9.7816	539.78	.29452	132.18	190,340	.20826	93.465	134,590

1·6	·0020951	11·062	477·30	·31416	140·99	203,030	·22214	99·697	143,560
1·7	·0023544	12·431	424·74	·33380	149·81	215,720	·23603	105·93	152,540
1·8	·0026274	13·873	380·60	·35343	158·62	228,410	·24991	112·16	161,510
1·9	·0029184	15·409	342·65	·37306	167·43	241,100	·26379	118·39	170,480
2·0	·0032238	17·022	310·19	·39270	176·24	253,790	·27768	124·62	179,450
2·1	·0035379	18·680	282·66	·41234	185·05	266,480	·29156	130·85	188,430
2·2	·0038647	20·406	258·75	·43197	193·87	279,160	·30544	137·08	197,400
2·3	·0042044	22·199	237·85	·45161	202·68	291,860	·31933	143·31	206,370
2·4	·0045564	24·057	219·47	·47124	211·49	304,540	·33321	149·54	215,340
2·5	·0049284	26·022	202·90	·49087	220·30	317,230	·34710	155·78	224,320
2·6	·0053137	28·056	188·19	·51051	229·11	329,920	·36098	162·01	233,290
2·7	·0057122	30·160	175·06	·53014	237·93	342,610	·37486	168·24	242,260
2·8	·0061238	32·333	163·30	·54978	246·74	355,300	·38875	174·47	251,230
2·9	·0065376	34·518	152·96	·56942	255·55	367,990	·40263	180·70	260,210
3·0	·0069738	36·821	143·39	·58905	264·36	380,680	·41651	186·93	269,180
3·1	·0074225	39·190	134·73	·60868	273·17	393,370	·43040	193·16	278,150
3·2	·0078837	41·626	126·84	·62832	281·99	406,060	·44428	199·39	287,120
3·3	·0083705	44·196	119·47	·64795	290·80	418,750	·45816	205·62	296,090
3·4	·0088569	46·764	112·91	·66759	299·61	431,440	·47205	211·86	305,070
3·5	·0093551	49·395	106·89	·68723	308·43	444,130	·48594	218·09	314,040

d = 6 *inches* = .5 *foot*

v	$s = \frac{h}{l}$	$s_m = 5280 \frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
3.6	.0098810	52.171	101.20	.70685	317.23	456,810	.49982	224.32	323,010
3.7	.010404	54.930	96.121	.72649	326.05	469,500	.51370	230.55	331,990
3.8	.010956	57.844	91.279	.74612	334.86	482,190	.52758	236.78	340,960
3.9	.011502	60.728	86.944	.76576	343.67	494,880	.54146	243.01	349,930
4.0	.012080	63.779	82.785	.78540	352.48	507,570	.55535	249.24	358,910
4.1	.012665	66.868	78.961	.80503	361.29	520,260	.56923	255.47	367,870
4.2	.013263	70.028	75.398	.82467	370.11	532,950	.58312	261.70	376,850
4.3	.013868	73.220	72.111	.84431	378.92	545,640	.59701	267.94	385,820
4.4	.014490	76.505	69.014	.86393	387.73	558,330	.61089	274.16	394,790
4.5	.015124	79.855	66.120	.88357	396.54	571,020	.62477	280.39	403,770
4.6	.015771	83.272	63.406	.90321	405.36	583,710	.63866	286.63	412,740
4.7	.016430	86.750	60.864	.92285	414.17	596,400	.65254	292.86	421,720
4.8	.017093	90.253	58.502	.94248	422.98	609,090	.66642	299.09	430,690
4.9	.017776	93.858	56.255	.96212	431.80	621,780	.68032	305.32	439,660
5.0	.018470	97.522	54.141	.98175	440.61	634,470	.69420	311.55	448,630

5.5	.022123	116.81	45.202	1.0799	484.66	697,910	.76361	342.70	493,490
6.0	.026059	137.59	38.374	1.1781	528.73	761,360	.83303	373.86	538,360
6.5	.030321	160.09	32.981	1.2763	572.78	824,800	.90244	405.01	583,220
7.0	.034861	184.07	28.685	1.3745	616.85	888,260	.97187	436.17	628,090
7.5	.039668	209.45	25.209	1.4726	660.91	951,700	1.0413	467.33	672,950
8.0	.044736	236.21	22.353	1.5708	704.97	1,015,100	1.1107	498.48	717,810
8.5	.050188	264.99	19.925	1.6690	749.03	1,078,600	1.1801	529.64	762,680
9.0	.055913	295.22	17.885	1.7671	793.09	1,142,000	1.2495	560.79	807,530
9.5	.061961	327.15	16.139	1.8653	837.14	1,205,500	1.3190	591.94	852,390
10.0	.068283	360.53	14.645	1.9635	881.21	1,268,900	1.3884	623.10	897,260
10.5	.074974	395.86	13.338	2.0617	925.27	1,332,400	1.4578	654.26	942,130
11.0	.081945	432.66	12.203	2.1598	969.33	1,395,800	1.5272	685.41	986,980
11.5	.089195	470.94	11.211	2.2580	1,013.4	1,459,300	1.5967	716.57	1,031,900
12.0	.096714	510.65	10.340	2.3562	1,057.5	1,522,700	1.6661	747.72	1,076,700
12.5	.10460	552.29	9.5600	2.4544	1,101.5	1,586,200	1.7355	778.88	1,121,600
13.0	.11277	595.42	8.8677	2.5525	1,145.6	1,649,600	1.8049	810.03	1,166,400
13.5	.12121	640.00	8.2499	2.6507	1,189.6	1,713,000	1.8743	841.18	1,211,300
14.0	.12994	686.06	7.6961	2.7489	1,233.7	1,776,500	1.9438	872.35	1,256,200
14.5	.13919	734.90	7.1846	2.8471	1,277.8	1,840,000	2.0132	903.50	1,301,000
15.0	.14874	785.33	6.7233	2.9452	1,321.8	1,903,400	2.0826	934.65	1,345,900

$d = 8$ *inches* $= .6667$ *foot*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q′		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.0000065670	.034674	152,280	.034907	15.666	22,559	.024683	11.077	15,951
.2	.000026045	.13751	38,395	.069814	31.332	45,118	.049365	22.155	31,903
.3	.000058180	.30719	17,188	.10472	46.998	67,677	.074047	33.232	47,854
.4	.00010269	.54218	9,738.5	.13963	62 664	90,236	.098730	44.310	63,806
.5	.00015904	.83975	6,287.5	.17453	78.330	112,800	.12341	55.387	79,757
.6	.00022735	1.2004	4,398.6	.20944	93.996	135,350	.14810	66.465	95,708
.7	.00030762	1.6242	3,250.7	.24435	109.66	157,910	.17278	77.543	111,660
.8	.00039880	2.1057	2,507.5	.27925	125.33	180,470	.19746	88.620	127,610
.9	.00050094	2.6450	1,996.2	.31416	140.99	203,030	.22214	99.697	143,560
1.0	.00061474	3.2458	1,626.7	.34907	156.66	225,590	.24683	110.77	159,510
1.1	.00073818	3.8975	1,354.7	.38397	172.33	248,150	.27151	121.85	175,460
1.2	.00087581	4.6242	1,141.8	.41888	187.99	270,710	.29619	132.93	191,420
1.3	.0010215	5.3936	978.93	.45378	203.66	293,260	.32087	144.01	207,370
1.4	.0011775	6.2170	849.28	.48870	219.33	315,830	.34556	155.09	223,320
1.5	.0013433	7.0923	744.46	.52360	234.99	338,380	.37024	166.16	239,270

1.6	.0015212	8.0317	657.39	.55851	250.66	360,940	.39492	177.24	255,220
1.7	.0017065	9.0101	586.00	.59342	266.32	383,500	.41960	188.32	271,180
1.8	.0019041	10.053	525.19	.62832	281.99	406,060	.44428	199.39	287,120
1.9	.0021148	11.166	472.87	.66322	297.65	428,620	.46896	210.47	303,070
2.0	.0023283	12.293	429.50	.69814	313.32	451,180	.49365	221.55	319,030
2.1	.0025546	13.488	391.44	.73304	328.99	473,740	.51833	232.63	334,980
2.2	.0027902	14.732	358.40	.76794	344.65	496,300	.54301	243.70	350,930
2.3	.0030398	16.050	328.97	.80286	360.32	518,860	.56770	254.78	366,890
2.4	.0032937	17.390	303.61	.83776	375.98	541,410	.59238	265.86	382,830
2.5	.0035622	18.808	280.72	.87267	391.65	563,980	.61706	276.94	398,790
2.6	.0038403	20.276	260.40	.90757	407.31	586,530	.64174	288.01	414,730
2.7	.0041209	21.758	242.66	.94248	422.98	609,090	.66642	299.09	430,690
2.8	.0044173	23.323	226.38	.97739	438.65	631,660	.69111	310.17	446,640
2.9	.0047228	24.936	211.74	1.0123	454.32	654,210	.71580	321.25	462,590
3.0	.0050372	26.596	198.52	1.0472	469.98	676,770	.74047	332.32	478,540
3.1	.0053697	28.352	186.23	1.0821	485.65	699,330	.76516	343.40	494,490
3.2	.0057026	30.179	175.36	1.1170	501.31	721,890	.78984	354.48	510,450
3.3	.0060544	31.967	165.17	1.1519	516.98	744,440	.81452	365.55	526,390
3.4	.0064055	33.821	156.12	1.1868	532.65	767,010	.83921	376.63	542,350
3.5	.0067763	35.778	147.57	1.2218	548.31	789,570	.86389	387.71	558,300

$d = 8$ *inches* $= .6667$ *foot*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
3.6	.0071568	37.788	139.73	1.2566	563.98	812,120	.88857	398.79	574,250
3.7	.0075344	39.781	132.72	1.2915	579.64	834,680	.91325	409.86	590,200
3.8	.0079338	41.890	126.04	1.3264	595.31	857,240	.93793	420.94	606,150
3.9	.0083284	43.973	120.07	1.3614	610.97	879,790	.96261	432.02	622,100
4.0	.0087462	46.180	114.34	1.3963	626.64	902,360	.98730	443.10	638,060
4.1	.0091692	48.413	109.06	1.4312	642.30	924,910	1.0120	454.17	654,000
4.2	.0096015	50.696	104.15	1.4661	657.98	947,480	1.0367	465.25	669,960
4.3	.010038	53.002	99.619	1.5010	673.64	970,040	1.0614	476.33	685,910
4.4	.010488	55.376	95.348	1.5359	689.30	992,590	1.0860	487.41	701,860
4.5	.010946	57.796	91.355	1.5708	704.97	1,015,100	1.1107	498.48	717,810
4.6	.011414	60.266	87.611	1.6057	720.64	1,037,700	1.1354	509.57	733,770
4.7	.011890	62.778	84.105	1.6406	736.31	1,060,300	1.1601	520.64	749,720
4.8	.012369	65.307	80.848	1.6755	751.97	1,082,800	1.1848	531.72	765,670
4.9	.012862	67.909	77.750	1.7104	767.64	1,105,400	1.2095	542.80	781,630
5.0	.013363	70.554	74.836	1.7453	783.30	1,128,000	1.2341	553.87	797,570

5.5	.016014	84.551	62.447	1.9199	861.65	1,240,700	1.3575	609.26	877,320
6.0	.018873	99.648	52.986	2.0944	939.96	1,353,500	1.4809	664.65	957,080
6.5	.021952	115.91	45.553	2.2689	1,018.3	1,466,300	1.6044	720.03	1,036,800
7.0	.025231	133.22	39.633	2.4435	1,096.6	1,579,100	1.7278	775.43	1,116,600
7.5	.028754	151.82	34.778	2.6180	1,175.0	1,691,900	1.8512	830.81	1,196,400
8.0	.032477	171.48	30.791	2.7925	1,253.3	1,804,700	1.9746	886.20	1,276,100
8.5	.036462	192.52	27.426	2.9671	1,331.6	1,917,500	2.0980	941.59	1,355,900
9.0	.040650	214.63	24.600	3.1416	1,409.9	2,030,300	2.2214	996.97	1,435,600
9.5	.044955	237.36	22.244	3.3161	1,488.3	2,143,100	2.3448	1,052.3	1,515,400
10.0	.049440	261.04	20.226	3.4907	1,566.6	2,255,900	2.4683	1,107.7	1,595,700
10.5	.054301	286.71	18.416	3.6652	1,644.9	2,368,700	2.5917	1,163.1	1,674,900
11.0	.059370	313.47	16.843	3.8397	1,723.3	2,481,500	2.7151	1,218.5	1,754,600
11.5	.064644	341.32	15.469	4.0143	1,801.6	2,594,300	2.8385	1,273.9	1,834,400
12.0	.070118	370.22	14.262	4.1888	1,879.9	2,707,100	2.9619	1,329.3	1,914,200
12.5	.075865	400.56	13.181	4.3633	1,958.3	2,819,900	3.0853	1,384.7	1,993,900
13.0	.081818	432.00	12.222	4.5378	2,036.6	2,932,600	3.2087	1,440.1	2,073,700
13.5	.087977	464.52	11.367	4.7124	2,114.9	3,045,400	3.3321	1,495.4	2,153,400
14.0	.094343	498.13	10.600	4.8870	2,193.3	3,158,300	3.4556	1,550.9	2,233,200
14.5	.10106	533.57	9.8956	5.0615	2,271.6	3,271,100	3.5790	1,606.2	2,313,000
15.0	.10799	570.16	9.2604	5.2360	2,349.9	3,383,800	3.7024	1,661.6	2,392,700

$d = 10$ *inches* $= .8333$ *foot*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.0000051045	.026951	195,910	.054542	24.478	35,248	.038566	17.309	24,924
.2	.000020239	.10686	49,411	.10908	48.956	70,497	.077133	34.617	49,848
.3	.000045201	.23866	22,123	.16363	73.434	105,750	.11570	51.925	74,772
.4	.00007976	.42113	12,537	.21817	97.913	140,990	.15427	69.234	99,697
.5	.00012369	.65309	8,084.6	.27271	122.39	176,240	.19283	86.543	124,620
.6	.00017704	.93478	5,648.3	.32725	146.87	211,490	.23140	103.85	149,540
.7	.00023915	1.2627	4,181.5	.38179	171.35	246,740	.26997	121.16	174,470
.8	.00031092	1.6416	3,216.3	.43633	195.83	281,990	.30853	138.47	199,390
.9	.00039169	2.0681	2,553.1	.49087	220.30	317,230	.34710	155.78	224,320
1.0	.00047984	2.5336	2,084.0	.54542	244.78	352,480	.38566	173.09	249,240
1.1	.00057790	3.0513	1,730.4	.59996	269.26	387,730	.42423	190.39	274,160
1.2	.00068237	3.6029	1,465.5	.65450	293.74	422,980	.46280	207.70	299,090
1.3	.00079706	4.2084	1,254.6	.70904	318.21	458,230	.50136	225.01	324,010
1.4	.00091857	4.8500	1,088.7	.76359	342.70	493,480	.53993	242.32	348,940
1.5	.0010494	5.5409	952.91	.81813	367.17	528,730	.57850	259.63	373,860

1.6	.0011883	6.2741	841.55	.87267	391.65	563,980	.61706	276.94	398,790
1.7	.0013328	7.0374	750.27	.92721	416.13	599,230	.65563	294.25	423,710
1.8	.0014870	7.8513	672.50	.98175	440.61	634,470	.69419	311.55	448,630
1.9	.0016487	8.7050	606.54	1.0363	465.08	669,710	.73276	328.86	473,550
2.0	.0018179	9.5984	550.09	1.0908	489.56	704,970	.77133	346.17	498,480
2.1	.0019960	10.539	501.00	1.1454	514.04	740,220	.80990	363.48	523,410
2.2	.0021825	11.523	458.19	1.1999	538.52	775,460	.84846	380.79	548,330
2.3	.0023746	12.538	421.12	1.2545	563.00	810,720	.88703	398.10	573,260
2.4	.0025737	13.589	388.54	1.3090	587.48	845,960	.92559	415.40	598,180
2.5	.0027798	14.677	359.73	1.3635	611.96	881,210	.96416	432.71	623,100
2.6	.0029940	15.808	334.00	1.4181	636.43	916,450	1.0027	450.02	648,020
2.7	.0032165	16.983	310.90	1.4726	660.91	951,700	1.0413	467.33	672,950
2.8	.0034446	18.187	290.31	1.5272	685.39	986,960	1.0799	484.64	697,880
2.9	.0036810	19.435	271.67	1.5817	709.87	1,022,200	1.1184	501.95	722,800
3.0	.0039223	20.710	254.95	1.6363	734.34	1,057,500	1.1570	519.25	747.720
3.1	.0041738	22.037	239.59	1.6908	758.82	1,092,700	1.1956	536.56	772,650
3.2	.0044322	23.402	225.62	1.7453	783.30	1,128,000	1.2341	553.87	797,570
3.3	.0046972	24.801	212.89	1.7999	807.77	1,163,200	1.2727	571.18	822,490
3.4	.0049690	26.236	201.25	1.8544	832.26	1,198,500	1.3113	588.49	847,420
3.5	.0052473	27.706	190.57	1.9090	856.74	1,233,700	1.3498	605.80	872,350

$d = 10$ *inches* $= .8333$ *foot*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q′		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
3.6	.0055394	29.248	180.53	1.9635	881.21	1,268,900	1.3884	623.10	897,260
3.7	.0058386	30.828	171.27	2.0180	905.69	1,304,200	1.4270	640.41	922,190
3.8	.0061477	32.459	162.66	2.0726	930.17	1,339,400	1.4655	657.72	947,110
3.9	.0064612	34.115	154.77	2.1271	954.64	1,374,700	1.5041	675.03	972,030
4.0	.0067820	35.809	147.45	2.1817	979.13	1,409,900	1.5427	692.34	996,970
4.1	.0071126	37.554	140.60	2.2362	1,003.6	1,445,200	1.5812	709.64	1,021,900
4.2	.0074509	39.341	134.21	2.2908	1,028.1	1,480,400	1.6198	726.96	1,046,800
4.3	.0077961	41.163	128.27	2.3453	1,052.6	1,515,700	1.6584	744.27	1,071,700
4.4	.0081484	43.023	122.72	2.3998	1,077.0	1,550,900	1.6969	761.57	1,096,700
4.5	.0085079	44.921	117.54	2.4544	1,101.5	1,586,200	1.7355	778.88	1,121,600
4.6	.0088746	46.858	112.68	2.5089	1,126.0	1,621,400	1.7741	796.20	1,146,500
4.7	.0092480	48.829	108.13	2.5635	1,150.5	1,656,700	1.8126	813.50	1,171,400
4.8	.0096285	50.838	103.86	2.6180	1,175.0	1,691,900	1.8512	830.81	1,196,400
4.9	010016	52.885	99.839	2.6726	1,199.4	1,727,200	1.8898	848.13	1,221,300
5.0	.010410	54.965	96.059	2.7271	1,223.9	1,762,400	1.9283	865.43	1,246,200

5·5	·012484	65·913	80·105	2·9998	1,346·3	1,938,700	2·1211	951·96	1,370,800
6·0	·014722	77·732	67·925	3·2725	1,468·7	2,114,900	2·3140	1,038·5	1,495,400
6·5	·017152	90·561	58·303	3·5452	1,591·1	2,291,100	2·5068	1,125·0	1,620,100
7·0	·019746	104·26	50·643	3·8179	1,713·5	2,467,400	2·6997	1,211·6	1,744,700
7·5	·022499	118·80	44·446	4·0906	1,835·9	2,643,600	2·8925	1,298·1	1,869,300
8·0	·025409	134·16	39·357	4·3633	1,958·3	2,819,900	3·0853	1,384·7	1,993,900
8·5	·028495	150·45	35·094	4·6361	2,080·7	2,996,100	3·2782	1,471·2	2,118,600
9·0	·031734	167·56	31·512	4·9087	2,203·0	3,172,300	3·4710	1,557·8	2,243,200
9·5	·035190	185·80	28·417	5·1814	2,325·4	3,348,600	3·6638	1,644·3	2,367,800
10·0	·038805	204·89	25·770	5·4542	2,447·8	3,524,800	3·8566	1,730·9	2,492,400
10·5	·042619	225·02	23·464	5·7269	2,570·2	3,701,100	4·0495	1,817·4	2,617,000
11·0	·046593	246·01	21·462	5·9996	2,692·6	3,877,300	4·2423	1,903·9	2,741,600
11·5	·050728	267·84	19·713	6·2723	2,815·0	4,053,600	4·4352	1,990·5	2,866,300
12·0	·055020	290·50	18·175	6·5450	2,937·4	4,229,800	4·6280	2,077·0	2,990,900
12·5	·059555	314·45	16·791	6·8177	3,059·8	4,406,100	4·8208	2,163·6	3,115,500
13·0	·064255	339·27	15·563	7·0904	3,182·1	4,582,300	5·0136	2,250·1	3,240,100
13·5	·069124	364·97	14·467	7·3631	3,304·5	4,758,500	5·2064	2,336·6	3,364,700
14·0	·074158	391·55	13·485	7·6359	3,427·0	4,934,800	5·3993	2,423·2	3,489,400
14·5	·079354	418·99	12·602	7·9086	3,549·4	5,111,000	5·5922	2,509·7	3,614,000
15·0	·084709	447·26	11·805	8·1813	3,671·7	5,287,300	5·7850	2,596·3	3,738,600

$d = 12$ *inches* $= 1$ *foot*

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
.1	.0000041604	.021967	240,360	.078540	35.248	50,757	.055535	24.924	35,891
.2	.000016468	.086948	60,725	.15708	70.497	101,520	.11107	49.848	71,781
.3	.000036772	.19416	27,194	.23562	105.75	152,270	.16661	74.772	107,670
.4	.000064875	.34254	15,414	.31416	140.99	203,030	.22214	99.697	143,560
.5	.00010075	.53194	9,925.9	.39270	176.24	253,790	.27768	124.62	179,450
.6	.00014373	.75889	6,957.4	.47124	211.49	304,540	.33321	149.54	215,340
.7	.00019442	1.0265	5,143.6	.54978	246.74	355,300	.38875	174.47	251,230
.8	.00025194	1.3302	3,969.3	.62832	281.99	406,060	.44428	199.39	287,120
.9	.00031684	1.6729	3,156.2	.70685	317.23	456,810	.49982	224.32	323,010
1.0	.00038805	2.0489	2,577.0	.78540	352.48	507,570	.55535	249.24	358,910
1.1	.00046653	2.4633	2,143.5	.86393	387.73	558,330	.61089	274.16	394,790
1.2	.00055254	2.9174	1,809.8	.94248	422.98	609,090	.66642	299.09	430,690
1.3	.00064424	3.4016	1,552.2	1.0210	458.23	659,840	.72195	324.01	466,570
1.4	.00074353	3.9258	1,344.9	1.0996	493.48	710,610	.77750	348.94	502,470
1.5	.00084934	4.4845	1,177.4	1.1781	528.73	761,360	.83303	373.86	538,360

1.6	.00096000	5.0687	1,041.7	1.2566	563.98	812,120	.88857	398.79	574,250
1.7	.0010801	5.7031	925.81	1.3352	599.23	862,880	.94410	423.71	610,140
1.8	.0012069	6.3725	828.55	1.4137	634.47	913,630	.99963	448.63	646,030
1.9	.0013402	7.0764	746.14	1.4922	669.71	964,380	1.0552	473.55	681,910
2.0	.0014751	7.7884	677.92	1.5708	704.97	1,015,100	1.1107	498.48	717,810
2.1	.0016208	8.5580	616.96	1.6493	740.22	1,065,900	1.1662	523.41	753,700
2.2	.0017728	9.3605	564.07	1.7279	775.46	1,116,700	1.2218	548.33	789,590
2.3	.0019295	10.188	518.27	1.8064	810.72	1,167,400	1.2773	573.26	825,490
2.4	.0020910	11.041	478.23	1.8850	845.96	1,218,200	1.3328	598.18	861,370
2.5	.0022582	11.923	442.82	1.9635	881.21	1,268,900	1.3884	623.10	897,260
2.6	.0024319	12.840	411.20	2.0420	916.45	1,319,700	1.4439	648.02	933,150
2.7	.0026124	13.793	382.79	2.1206	951.70	1,370,400	1.4994	672.95	969,040
2.8	.0027986	14.776	357.32	2.1991	986.96	1,421,200	1.5550	697.88	1,004,900
2.9	.0029916	15.796	334.26	2.2777	1,022.2	1,472,000	1.6105	722.80	1,040,800
3.0	0031902	16.844	313.46	2.3562	1,057.5	1,522,700	1.6661	747.72	1,076,700
3.1	.0033960	17.931	294.46	2.4347	1,092.7	1,573,500	1.7216	772.65	1,112,600
3.2	.0036075	19.048	277.20	2.5133	1,128.0	1,624,200	1.7771	797.57	1,148,500
3.3	.0038229	20.185	261.58	2.5918	1,163.2	1,675,000	1.8327	822.49	1,184,400
3.4	.0040457	21.361	247.18	2.6704	1,198.5	1,725,800	1.8882	847.42	1,220,300
3.5	.0042738	22.565	233.99	2.7489	1,233.7	1,776,500	1.9437	872.35	1,256,200

$d = 12$ *inches* $= 1$ *foot*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q′		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
3.6	.0045093	23.809	221.76	2.8274	1,268.9	1,827,300	1.9993	897.26	1,292,100
3.7	.0047484	25.071	210.60	2.9060	1,304.2	1,878,000	2.0548	922.19	1,327,900
3.8	.0049951	26.374	200.20	2.9845	1,339.4	1,928,800	2.1103	947.11	1,363,800
3.9	.0052448	27.692	190.66	3.0630	1,374.7	1,979,500	2.1659	972.03	1,399,700
4.0	.0055025	29.053	181.74	3.1416	1,409.9	2,030,300	2.2214	996.97	1,435,600
4.1	.0057705	30.468	173.30	3.2201	1,445.2	2,081,000	2.2769	1,021.9	1,471,500
4.2	.0060445	31.915	165.44	3.2987	1,480.4	2,131,800	2.3325	1,046.8	1,507,400
4.3	.0063243	33.392	158.12	3.3772	1,515.7	2,182,600	2.3880	1,071.7	1,543,300
4.4	.0066097	34.899	151.29	3.4557	1,550.9	2,233,300	2.4435	1,096.7	1,579,200
4.5	.0069010	36.437	144.91	3.5343	1,586.2	2,284,100	2.4991	1,121.6	1,615,100
4.6	.0071981	38.006	138.92	3.6129	1,621.4	2,334,900	2.5546	1,146.5	1,651,000
4.7	.0075007	39.603	133.32	3.6914	1,656.7	2,385,600	2.6102	1,171.4	1,686,900
4.8	.0078089	41.231	128.06	3.7699	1,691.9	2,436,400	2.6657	1,196.4	1,722,700
4.9	.0081229	42.888	123.11	3.8485	1,727.2	2,487,100	2.7213	1,221.3	1,758,700
5.0	.0084421	44.574	118.45	3.9270	1,762.4	2,537,900	2.7768	1,246.2	1,794,500

5.5	.010130	53.487	98.714	4.3197	1,938.7	2,791,600	3.0544	1,370.8	1,974,000
6.0	.011955	63.122	83.647	4.7124	2,114.9	3,045,400	3.3321	1,495.4	2,153,400
6.5	.013939	73.595	71.743	5.1051	2,291.1	3,299,200	3.6098	1,620.1	2,332,900
7.0	.016059	84.791	62.270	5.4978	2,467.4	3,553,000	3.8875	1,744.7	2,512,300
7.5	.018313	96.690	54.607	5.8905	2,643.6	3,806,800	4.1651	1,869.3	2,691,800
8.0	.020696	109.27	48.318	6.2832	2,819.9	4,060,600	4.4428	1,993.9	2,871,200
8.5	.023185	122.41	43.132	6.6759	2,996.1	4,314,400	4.7205	2,118.6	3,050,700
9.0	.025791	136.17	38.774	7.0685	3,172.3	4,568,100	4.9982	2,243.2	3,230,100
9.5	.028623	151.13	34.937	7.4612	3,348.6	4,821,900	5.2758	2,367.8	3,409,600
10.0	.031591	166.80	31.654	7.8540	3,524.8	5,075,700	5.5535	2,492.4	3,589,100
10.5	.034693	183.18	28.824	8.2467	3,701.1	5,329,500	5.8312	2,617.0	3,768,500
11.0	.037925	200.24	26.368	8.6393	3,877.3	5,583,300	6.1089	2,741.6	3,947,900
11.5	.041287	217.99	24.221	9.0321	4,053.6	5,837,100	6.3866	2,866.3	4,127,400
12.0	.044775	236.41	22.334	9.4248	4,229.8	6,090,900	6.6642	2,990.9	4,306,900
12.5	.048440	255.76	20.644	9.8175	4,406.1	6,344,700	6.9419	3,115.5	4,486,300
13.0	.052234	275.79	19.145	10.210	4,582.3	6,598,400	7.2195	3,240.1	4,665,700
13.5	.056158	296.51	17.807	10.603	4,758.5	6,852,200	7.4972	3,364.7	4,845,200
14.0	.060214	317.93	16.607	10.996	4,934.8	7,106,100	7.7750	3,489.4	5,024,700
14.5	.064494	340.53	15.505	11.388	5,111.0	7,359,900	8.0527	3,614.0	5,204,200
15.0	.068913	363.86	14.511	11.781	5,287.3	7,613,600	8.3303	3,738.6	5,383,600

$d = 14$ *inches* $= 1.1667$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5.280\frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
.1	.0000034647	.018294	288,620	.10690	47.977	69,086	.075589	33.924	48,851
.2	.000013731	.072500	72,827	.21380	95.953	138,170	.15118	67.848	97,701
.3	.000030703	.16211	32,570	.32070	143.93	207,260	.22677	101.77	146,550
.4	.000054072	.28550	18,494	.42760	191.91	276,340	.30236	135.70	195,400
.5	.000083954	.44327	11,911	.53450	239.88	345,430	.37795	169.62	244,250
.6	.00011993	.63324	8,338.0	.64140	287.86	414,510	.45353	203.54	293,100
.7	.00016220	.85641	6,165.2	.74831	335.84	483,600	.52913	237.47	341,960
.8	.00021049	1.1114	4,750.8	.85520	383.81	552,690	.60471	271.39	390,800
.9	.00026423	1.3951	3,784.5	.96210	431.79	621,770	.68030	305.32	439,650
1.0	.00032462	1.7140	3,080.5	1.0690	479.77	690,860	.75589	339.24	488,510
1.1	.00039021	2.0603	2,562.7	1.1759	527.74	759,940	.83148	373.16	537,350
1.2	.00046131	2.4357	2,167.8	1.2828	575.72	829,030	.90707	407.09	586,210
1.3	.00053869	2.8443	1,856.4	1.3897	623.69	898,110	.98265	441.01	635,050
1.4	.00061954	3.2711	1,614.1	1.4966	671.68	967,210	1.0583	474.94	683,910
1.5	.00070760	3.7361	1,413.2	1.6035	719.65	1,036,300	1.1338	508.86	732,760

1.6	.00080101	4.2293	1,248.4	1.7104	767.63	1,105,400	1.2094	542.79	781,610
1.7	.00089964	4.7501	1,111.6	1.8173	815.61	1,174,500	1.2850	576.71	830,460
1.8	.0010051	5.3070	994.90	1.9242	863.57	1,243,500	1.3606	610.63	879,310
1.9	.0011141	5.8826	897.55	2.0311	911.55	1,312,600	1.4362	644.56	928,150
2.0	.0012281	6.4844	814.25	2.1380	959.53	1,381,700	1.5118	678.48	977,010
2.1	.0013481	7.1180	741.77	2.2449	1,007.5	1,450,800	1.5874	712.41	1,025,900
2.2	.0014738	7.7814	678.53	2.3518	1,055.5	1,519,900	1.6630	746.33	1,074,700
2.3	.0016038	8.4678	623.53	2.4587	1,103.5	1,589,000	1.7386	780.26	1,123,600
2.4	.0017393	9.1835	574.94	2.5656	1,151.4	1,658,100	1.8141	814.18	1,172,400
2.5	.0018790	9.9209	532.21	2.6725	1,199.4	1,727,100	1.8897	848.11	1,221,300
2.6	.0020251	10.692	493.81	2.7794	1,247.4	1,796,200	1.9653	882.02	1,270,100
2.7	.0021761	11.489	459.55	2.8863	1,295.4	1,865,300	2.0409	915.95	1,319,000
2.8	.0023319	12.312	428.83	2.9932	1,343.4	1,934,400	2.1165	949.88	1,367,800
2.9	.0024925	13.160	401.21	3.1001	1,391.3	2,003,500	2.1921	983.81	1,416,700
3.0	.0026577	14.033	376.26	3.2070	1,439.3	2,072,600	2.2677	1,017.7	1,465,500
3.1	.0028302	14.943	353.34	3.3139	1,487.3	2,141,700	2.3433	1,051.6	1,514,400
3.2	.0030089	15.887	332.35	3.4208	1,535.3	2,210,800	2.4189	1,085.6	1,563,200
3.3	.0031912	16.849	313.36	3.5277	1,583.2	2,279,800	2.4944	1,119.5	1,612,100
3.4	.0033799	17.846	295.87	3.6346	1,631.2	2,348,900	2.5700	1,153.4	1,660,900
3.5	.0035718	18.859	279.97	3.7415	1,679.2	2,418,000	2.6456	1,187.4	1,709,800

d = 14 inches = 1.1667 feet

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q			Q′		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day.
3·6	.0037683	19·897	265·37	3·8484	1,727·1	2,487,100	2·7212	1,221·3	1,758,600
3·7	.0039715	20·970	251·79	3·9553	1,775·1	2,556,200	2·7968	1,255·2	1,807,500
3·8	.0041775	22·057	239·38	4·0622	1,823·1	2,625,200	2·8724	1,289·1	1,856,300
3·9	.0043902	23·180	227·78	4·1691	1,871·1	2,694,300	2·9479	1,323·0	1,905,200
4·0	.0046054	24·316	217·14	4·2760	1,919·1	2,763,400	3·0236	1,357·0	1,954,000
4·1	.0048296	25·500	207·06	4·3829	1,967·0	2,832,500	3·0991	1,390·9	2,002,900
4·2	.0050587	26·710	197·68	4·4898	2,015·0	2,901,600	3·1748	1,424·8	2,051,700
4·3	.0052926	27·945	188·94	4·5967	2,063·0	2,970,700	3·2503	1,458·7	2,100,600
4·4	.0055312	29·205	180·79	4·7036	2,111·0	3,039,800	3·3259	1,492·7	2,149,400
4·5	.0057747	30·490	173·17	4·8105	2,158·9	3,108,900	3·4015	1,526·6	2,198,300
4·6	.0060231	31·802	166·03	4·9175	2,206·9	3,178,000	3·4771	1,560·5	2,247,100
4·7	.0062761	33·138	159·33	5·0244	2,254·9	3,247,100	3·5527	1,594·4	2,296,000
4·8	.0065336	34·497	153·06	5·1312	2,302·9	3,316,100	3·6283	1,628·4	2,344,800
4·9	.0067959	35·882	147·15	5·2382	2,350·9	3,385,200	3·7039	1,662·3	2,393,700
5·0	.0070629	37·292	141·59	5·3450	2,398·8	3,454,300	3·7795	1,696·2	2,442,500

5·5	·0084653	44·696	118·13	5·8795	2,638·7	3,799,700	4·1574	1,865·8	2,686,800
6·0	·0099784	52·685	100·22	6·4140	2,878·6	4,145,100	4·5353	2,035·4	2,931,000
6·5	·011621	61·356	86·054	6·9485	3,118·5	4,490,600	4·9133	2,205·1	3,175,300
7·0	·013373	70·609	74·777	7·4831	3,358·4	4,836,000	5·2913	2,374·7	3,419,600
7·5	·015217	80·343	65·717	8·0175	3,598·2	5,181,400	5·6692	2,544·3	3,663,800
8·0	·017160	90·602	58·276	8·5520	3,838·1	5,526,900	6·0471	2,713·9	3,908,000
8·5	·019275	101·77	51·880	9·0866	4,078·0	5,872,300	6·4251	2,883·6	4,152,300
9·0	·021502	113·53	46·508	9·6210	4,317·9	6,217,700	6·8030	3,053·2	4,396,500
9·5	·023836	125·85	41·953	10·155	4,557·7	6,563,100	7·1809	3,222·8	4,640,800
10·0	·026279	138·75	38·053	10·690	4,797·7	6,908,600	7·5589	3,392·4	4,885,100
10·5	·028884	152·51	34·621	11·225	5,037·6	7,254,000	7·9369	3,562·1	5,129,300
11·0	·031604	166·87	31·642	11·759	5,277·4	7,599,400	8·3148	3,731·6	5,373,500
11·5	·034437	181·82	29·039	12·294	5,517·3	7,944,900	8·6928	3,901·3	5,617,800
12·0	·037381	197·37	26·752	12·828	5,757·2	8,290,300	9·0707	4,070·9	5,862,100
12·5	·040457	213·61	24·718	13·363	5,997·1	8,635,700	9·4487	4,240·5	6,106,300
13·0	·043645	230·44	22·912	13·897	6,236·9	8,981,100	9·8265	4,410·1	6,350,500
13·5	·046945	247·87	21·301	14·431	6,476·8	9,326,500	10·204	4,579·7	6,594,800
14·0	·050358	265·89	19·858	14·966	6,716·8	9,672,100	10·583	4,749·4	6,839,100
14·5	·053963	284·92	18·531	15·501	6,956·6	10,018,000	10·960	4,919·0	7,083,400
15·0	·057689	304·59	17·334	16·035	7,196·5	10,363,000	11·338	5,088·6	7,327,600

$d = 16$ *inches* $= 1.3333$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q′		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.0000029058	.015342	344,140	.13963	62.666	90,238	.098732	44.411	63,807
.2	.000011548	.060975	86,592	.27926	125.33	180,480	.19746	88.622	127,610
.3	.000025774	.13608	38,799	.41889	188.00	270,710	.29620	132.93	191,420
.4	.000045596	.24075	21,932	.55852	250.66	360,950	.39493	177.24	255,230
.5	.000070778	.37371	14,129	.69815	313.33	451,190	.49366	221.55	319,040
.6	.00010125	.53459	9,876.7	.83778	375.99	541,430	.59239	265.86	382,840
.7	.00013713	.72402	7,292.6	.97742	438.66	631,670	.69113	310.18	446,650
.8	.00017821	.94091	5,611.5	1.1170	501.33	721,900	.78986	354.49	510,460
.9	.00022403	1.1829	4,463.8	1.2567	563.99	812,140	.88859	398.80	574,260
1.0	.00027425	1.4480	3,646.3	1.3963	626.66	902,380	.98732	443.11	638,070
1.1	.00032958	1.7402	3,034.2	1.5359	689.32	992,610	1.0861	487.42	701,880
1.2	.00039022	2.0603	2,562.7	1.6756	751.99	1,082,900	1.1848	531.73	765,680
1.3	.00045560	2.4055	2,194.9	1.8152	814.65	1,173,100	1.2835	576.04	829,490
1.4	.00052565	2.7754	1,902.3	1.9548	877.32	1,263,300	1.3823	620.35	893,310
1.5	.00060027	3.1694	1,665.9	2.0945	939.98	1,353,600	1.4810	664.66	957,110

1.6	.00067939	3.5872	1,471.9	2.2341	1,002.7	1,443,800	1.5797	708.97	1,020,900
1.7	.00076294	4.0283	1,310.7	2.3737	1,065.3	1,534,100	1.6785	753.29	1,084,700
1.8	.00085079	4.4921	1,175.4	2.5133	1,128.0	1,624,300	1.7772	797.59	1,148,500
1.9	.00094456	4.9872	1,058.7	2.6530	1,190.6	1,714,500	1.8759	841.90	1,212,300
2.0	.0010429	5.5064	958.87	2.7926	1,253.3	1,804,800	1.9746	886.22	1,276,100
2.1	.0011447	6.0438	873.61	2.9322	1,316.0	1,895,000	2.0734	930.53	1,340,000
2.2	.0012512	6.6062	799.25	3.0718	1,378.6	1,985,200	2.1721	974.83	1,403,800
2.3	.0013620	7.1912	734.23	3.2115	1,441.3	2,075,500	2.2709	1,019.2	1,467,600
2.4	.0014776	7.8015	676.78	3.3511	1,504.0	2,165,700	2.3696	1,063.5	1,531,400
2.5	.0015975	8.4345	625.99	3.4908	1,566.6	2,256,000	2.4683	1,107.8	1,595,200
2.6	.0017215	9.0893	580.90	3.6304	1,629.3	2,346,200	2.5670	1,152.1	1,659,000
2.7	.0018497	9.7661	540.64	3.7700	1,692.0	2,436,400	2.6658	1,196.4	1,722,800
2.8	.0019819	10.465	504.56	3.9097	1,754.7	2,526,700	2.7645	1,240.7	1,786,600
2.9	.0021182	11.184	472.11	4.0493	1,817.3	2,616,900	2.8633	1,285.0	1,850,400
3.0	.0022583	11.924	442.80	4.1889	1,880.0	2,707,100	2.9620	1,329.3	1,914,200
3.1	.0024036	12.691	416.04	4.3285	1,942.6	2,797,400	3.0607	1,373.6	1,978,000
3.2	.0025528	13.479	391.72	4.4682	2,005.3	2,887,600	3.1594	1,417.9	2,041,800
3.3	.0027046	14.280	369.73	4.6078	2,068.0	2,977,800	3.2581	1,462.2	2,105,600
3.4	.0028617	15.109	349.45	4.7474	2,130.6	3,068,100	3.3569	1,506.6	2,169,500
3.5	.0030225	15.959	330.85	4.8871	2,193.3	3,158,300	3.4557	1,550.9	2,233,300

$d = 16$ *inches* $= 1.3333$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
3.6	.0031915	16.851	313.33	5.0267	2,256.0	3,248,600	3.5543	1,595.2	2,297,000
3.7	.0033650	17.767	297.18	5.1663	2,318.6	3,338,800	3.6531	1,639.5	2,360,900
3.8	.0035426	18.705	282.28	5.3059	2,381.3	3,429,000	3.7518	1,683.8	2,424,700
3.9	.0037243	19.664	268.50	5.4455	2,443.9	3,519,200	3.8505	1,728.1	2,488,500
4.0	.0039104	20.647	255.73	5.5852	2,506.6	3,609,500	3.9493	1,772.4	2,552,300
4.1	.0041004	21.650	243.88	5.7248	2,569.3	3,699,700	4.0480	1,816.7	2,616,100
4.2	.0042968	22.687	232.73	5.8645	2,632.0	3,790,000	4.1468	1,861.1	2,679,900
4.3	.0044953	23.735	222.45	6.0041	2,694.6	3,880,300	4.2455	1,905.4	2,743,700
4.4	.0046999	24.815	212.77	6.1437	2,757.3	3,970,500	4.3442	1,949.7	2,807,500
4.5	.0049066	25.907	203.81	6.2833	2,819.9	4,060,700	4.4229	1,994.0	2,871,300
4.6	.0051173	27.019	195.42	6.4230	2,882.6	4,151,000	4.5417	2,038.3	2,935,100
4.7	.0053345	28.166	187.46	6.5627	2,945.3	4,241,200	4.6404	2,082.6	2,999,000
4.8	.0055530	29.320	180.08	6.7022	3,007.9	4,331,400	4.7391	2,126.9	3,062,700
4.9	.0057786	30.511	173.05	6.8420	3,070.6	4,421,700	4.8379	2,171.3	3,126,600
5.0	.0060051	31.707	166.53	6.9815	3,133.3	4,511,900	4.9366	2,215.5	3,190,400

5.5	.0072096	38.066	138.70	7.6796	3,446.6	4,963,100	5.4303	2,437.1	3,509,400
6.0	.0085129	44.948	117.47	8.3778	3,759.9	5,414,300	5.9239	2,658.6	3,828,400
6.5	.0099218	52.387	100.79	9.0759	4,073.2	5,865,400	6.4176	2,880.2	4,147,400
7.0	.011427	60.335	87.510	9.7742	4,386.6	6,316,700	6.9113	3,101.8	4,466,500
7.5	.013000	68.637	76.925	10.472	4,699.9	6,767,800	7.4049	3,323.3	4,785,500
8.0	.014657	77.386	68.229	11.170	5,013.3	7,219,000	7.8986	3,544.9	5,104,600
8.5	.016512	87.185	60.561	11.869	5,326.6	7,670,300	8.3923	3,766.4	5,423,600
9.0	.018474	97.542	54.130	12.567	5,639.9	8,121,400	8.8859	3,988.0	5,742,600
9.5	.020457	108.01	48.882	13.265	5,953.2	8,572,500	9.3795	4,209.5	6,061,600
10.0	.022528	118.95	44.389	13.963	6,266.6	9,023,800	9.8732	4,431.1	6,380,700
10.5	.024773	130.80	40.367	14.661	6,579.9	9,475,000	10.367	4,652.6	6,699,800
11.0	.027117	143.18	36.877	15.359	6,893.2	9,926,100	10.861	4,874.2	7,018,800
11.5	.029562	156.08	33.827	16.058	7,206.6	10,377,000	11.354	5,095.8	7,337,900
12.0	.032104	169.51	31.149	16.756	7,519.9	10,829,000	11.848	5,317.3	7,656,800
12.5	.034744	183.45	28.782	17.454	7,833.2	11,280,000	12.342	5,538.9	7,975,900
13.0	.037480	197.89	26.681	18.152	8,146.5	11,731,000	12.835	5,760.4	8,294,900
13.5	.040313	212.85	24.806	18.850	8,459.8	12,182,000	13.329	5,981.9	8,613,900
14.0	.043240	228.31	23.127	19.548	8,773.2	12,633,000	13.823	6,203.5	8,933,100
14.5	.046311	243.96	21.593	20.247	9,086.6	13,085,000	14.316	6,425.1	9,252,100
15.0	.049480	261.25	20.210	20.945	9,399.8	13,536,000	14.810	6,646.6	9,571,100

d = 18 *inches* = 1.5 *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q: Cubic Feet per Second	Q: Gallons per Minute	Q: Gallons per Day	Q′: Cubic Feet per Second	Q′: Gallons per Minute	Q′: Gallons per Day
.1	.0000025000	.013200	400,000	.17671	79.309	114,200	.12495	56.079	80,753
.2	.0000099502	.052536	100,500	.35343	158.62	228,410	.24991	112.16	161,510
.3	.000022276	.11761	44,892	.53014	232.92	342,610	.37486	168.24	242,260
.4	.000039270	.20734	25,465	.70685	317.23	456,810	.49982	224.32	323,010
.5	.000060945	.32179	16,408	.88357	396.54	571,020	.62477	280.39	403,770
.6	.000087163	.46021	11,473	1.0603	475.85	685,220	.74972	336.47	484,520
.7	.00011783	.62213	8,486.9	1.2370	555.16	799,430	.87468	392.55	565,280
.8	.00015310	.80837	6,531.6	1.4137	634.47	913,630	.99963	448.63	646,030
.9	.00019242	1.0160	5,197.0	1.5904	713.77	1,027,800	1.1246	504.71	726,780
1.0	.00023631	1.2477	4,231.7	1.7671	793.09	1,142,000	1.2495	560.79	807,530
1.1	.00028494	1.5045	3,509.5	1.9438	872.39	1,256,200	1.3745	616.86	888,280
1.2	.00033671	1.7778	2,969.9	2.1206	951.70	1,370,400	1.4994	672.95	969,040
1.3	.00039306	2.0753	2,544.1	2.2973	1,031.0	1,484,600	1.6244	729.02	1,049,800
1.4	.00045424	2.3984	2,201.5	2.4740	1,110.3	1,598,900	1.7494	785.11	1,130,600
1.5	.00051864	2.7384	1,928.1	2.6507	1,189.6	1,713,000	1.8743	841.18	1,211,300

1.6	.00058694	3.0990	1,703.8	2.8274	1,268.9	1,827,300	1.9993	897.26	1,292,100
1.7	.00066019	3.4858	1,514.7	3.0041	1,348.2	1,941,500	2.1242	953.34	1,372,800
1.8	.00073745	3.8937	1,356.0	3.1808	1,427.5	2,055,700	2.2492	1,009.4	1,453,600
1.9	.00081717	4.3146	1,223.7	3.3575	1,506.9	2,169,900	2.3741	1,065.5	1,534,300
2.0	.00090049	4.7546	1,110.5	3.5343	1,586.2	2,284,100	2.4991	1,121.6	1,615,100
2.1	.00098915	5.2226	1,011.0	3.7110	1,665.5	2,398,300	2.6240	1,177.7	1,695,800
2.2	.0010816	5.7106	924.59	3.8877	1,744.8	2,512,500	2.7490	1,233.7	1,776,600
2.3	.0011777	6.2184	849.08	4.0644	1,824.1	2,626,700	2.8740	1,289.8	1,857,300
2.4	.0012776	6.7456	782.73	4.2411	1,903.4	2,740,900	2.9989	1,345.9	1,938,100
2.5	.0013811	7.2922	724.05	4.4178	1,982.7	2,855,100	3.1238	1,402.0	2,018,800
2.6	.0014882	7.8574	671.97	4.5945	2,062.0	2,969,300	3.2488	1,458.0	2,099,600
2.7	.0015988	8.4417	625.46	4.7712	2,141.3	3,083,500	3.3737	1,514.1	2,180,300
2.8	.0017130	9.0444	583.78	4.9480	2,220.6	3,197,700	3.4987	1,570.2	2,261,100
2.9	.0018305	9.6652	546.29	5.1247	2,300.0	3,311,900	3.6237	1,626.3	2,341,900
3.0	.0019515	10.304	512.44	5.3014	2,379.2	3,426,100	3.7486	1,682.4	2,422,600
3.1	.0020777	10.970	481.29	5.4781	2,458.6	3,540,300	3.8736	1,738.4	2,503,300
3.2	.0022087	11.662	452.76	5.6548	2,537.9	3,654,500	3.9985	1,794.5	2,584,100
3.3	.0023421	12.366	426.97	5.8315	2,617.2	3,768,700	4.1234	1,850.6	2,664,800
3.4	.0024802	13.095	403.19	6.0083	2,696.5	3,882,900	4.2484	1,906.7	2,745,600
3.5	.0026207	13.837	381.58	6.1850	2,775.8	3,997,100	4.3734	1,962.8	2,826,400

d = 18 *inches* = 1.5 *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
3.6	.0027671	14.610	361.38	6.3617	2,855.1	4,111,300	4.4983	2,018.8	2,907,100
3.7	.0029173	15.403	342.78	6.5384	2,934.4	4,225,500	4.6233	2,074.9	2,987,900
3.8	.0030711	16.215	325.61	6.7151	3,013.7	4,339,700	4.7482	2,131.0	3,068,600
3.9	.0032286	17.047	309.73	6.8918	3,093.0	4,453,900	4.8732	2,187.1	3,149,300
4.0	.0033897	17.897	295.01	7.0685	3,172.3	4,568,100	4.9982	2,243.2	3,230,100
4.1	.0035543	18.766	281.35	7.2452	3,251.6	4,682,300	5.1231	2,299.2	3,310,900
4.2	.0037225	19.655	268.63	7.4220	3,331.0	4,796,600	5.2481	2,355.3	3,391,600
4.3	.0038942	20.561	256.79	7.5987	3,410.3	4,910,800	5.3730	2,411.4	3,472,400
4.4	.0040694	21.486	245.74	7.7754	3,489.6	5,024,900	5.4979	2,467.5	3,553,100
4.5	.0042481	22.430	235.40	7.9521	3,568.9	5,139,100	5.6229	2,523.5	3,633,900
4.6	.0044303	23.392	225.72	8.1289	3,648.2	5,253,400	5.7479	2,579.6	3,714,700
4.7	.0046158	24.371	216.65	8.3056	3,727.5	5,367,600	5.8729	2,635.7	3,795,400
4.8	.0048047	25.369	208.13	8.4822	3,806.8	5,481,800	5.9978	2,691.8	3,876,100
4.9	.0049971	26.385	200.12	8.6590	3,886.2	5,596,000	6.1228	2,747.9	3,956,900
5.0	.0051928	27.418	192.58	8.8357	3,965.4	5,710,200	6.2477	2,803.9	4,037,700

5.5	.0062455	32.976	160.11	9.7192	4,361.9	6,281,200	6.8724	3,084.3	4,441,400
6.0	.0073881	39.009	135.35	10.603	4,758.5	6,852,200	7.4972	3,364.7	4,845,200
6.5	.0086181	45.503	116.04	11.486	5,155.0	7,423,200	8.1219	3,645.1	5,248,900
7.0	.0099341	52.452	100.66	12.370	5,551.6	7,994,300	8.7468	3,925.5	5,652,800
7.5	.011334	59.843	88.231	13.253	5,948.1	8,565,200	9.3715	4,205.9	6,056,500
8.0	.012816	67.667	78.028	14.137	6,344.7	9,136,300	9.9963	4,486.3	6,460,300
8.5	.014393	75.994	69.478	15.021	6,741.2	9,707,300	10.621	4,766.7	6,864,000
9.0	.016052	84.754	62.297	15.904	7,137.7	10,278,000	11.246	5,047.1	7,267,800
9.5	.017791	93.938	56.207	16.788	7,534.3	10,849,000	11.871	5,327.5	7,671,500
10.0	.019610	103.54	50.994	17.671	7,930.9	11,420,000	12.495	5,607.9	8,075,300
10.5	.021563	113.85	46.376	18.555	8,327.4	11,991,000	13.120	5,888.3	8,479,100
11.0	.023603	124.62	42.368	19.438	8,723.9	12,562,000	13.745	6,168.6	8,882,800
11.5	.025729	135.85	38.867	20.322	9,120.5	13,133,000	14.370	6,449.1	9,286,700
12.0	.027940	147.52	35.791	21.206	9,517.0	13,704,000	14.994	6,729.5	9,690,400
12.5	.030236	159.64	33.073	22.089	9,913.6	14,275,000	15.619	7,009.9	10,094,000
13.0	.032615	172.21	30.661	22.973	10,310	14,846,000	16.244	7,290.2	10,498,000
13.5	.035078	185.21	28.508	23.856	10,707	15,417,000	16.869	7,570.6	10,902,000
14.0	.037624	198.65	26.579	24.740	11,103	15,989,000	17.494	7,851.1	11,306,000
14.5	.040294	212.75	24.818	25.624	11,500	16,560,000	18.118	8,131.5	11,709,000
15.0	.043050	227.30	23.229	26.507	11,896	17,130,000	18.743	8,411.8	12,113,000

$d = 20$ *inches* $= 1.6667$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.0000021903	.011565	456,560	.21817	97.913	140,990	.15427	69.234	99,697
.2	.0000087014	.045943	114,920	.43633	195.83	281,990	.30853	138.47	199,390
.3	.000019410	.10248	51,519	.65450	293.74	422,980	.46280	207.70	299,090
.4	.000034268	.18093	29,182	.87267	391.65	563,980	.61706	276.94	398,790
.5	.000053170	.28074	18,807	1.0908	489.56	704,970	.77133	346.17	498,480
.6	.000076162	.40213	13,130	1.3090	587.48	845,960	.92559	415.40	598,180
.7	.00010294	.54350	9,714.7	1.5272	685.39	986,960	1.0799	484.64	697,880
.8	.00013397	.70734	7,464.5	1.7453	783.30	1,128,000	1.2341	553.87	797,570
.9	.00016864	.89043	5,929.7	1.9635	881.21	1,268,900	1.3884	623.10	897,260
1.0	.00020709	1.0934	4,828.9	2.1817	979.13	1,409,900	1.5427	692.34	996,970
1.1	.00024922	1.3159	4,012.5	2.3998	1,077.0	1,550,900	1.6969	761.57	1,096,700
1.2	.00029552	1.5603	3,383.9	2.6180	1,175.0	1,691,900	1.8512	830.81	1,196,400
1.3	.00034556	1.8245	2,893.9	2.8362	1,272.9	1,832,900	2.0054	900.04	1,296,000
1.4	.00039858	2.1045	2,508.9	3.0544	1,370.8	1,973,900	2.1597	969.28	1,395,800
1.5	.00045503	2.4025	2,197.7	3.2725	1,468.7	2,114,900	2.3140	1,038.5	1,495,400

1.6	.00051486	2.7184	1,942.3	3.4907	1,566.6	2,255,900	2.4683	1,107.7	1,595,100
1.7	.00057799	3.0518	1,730.1	3.7089	1,664.5	2,396,900	2.6225	1,177.0	1,694,800
1.8	.00064557	3.4086	1,549.0	3.9270	1,762.4	2,537,900	2.7768	1,246.2	1,794,500
1.9	.00071659	3.7836	1,395.5	4.1451	1,860.3	2,678,900	2.9310	1,315.4	1,894,200
2.0	.00078955	4.1688	1,266.5	4.3633	1,958.3	2,819,900	3.0853	1,384.7	1,993,900
2.1	.00086718	4.5787	1,153.2	4.5815	2,056.2	2,960,900	3.2396	1,453.9	2,093,600
2.2	.00094811	5.0060	1,054.7	4.7997	2,154.1	3,101,800	3.3938	1,523.1	2,193,300
2.3	.0010318	5.4480	969.15	5.0179	2,252.0	3,242,900	3.5481	1,592.4	2,293,000
2.4	.0011187	5.9065	893.92	5.2360	2,349.9	3,383,800	3.7024	1,661.6	2,392,700
2.5	.0012080	6.3782	827.81	5.4542	2,447.8	3,524,800	3.8566	1,730.9	2,492,400
2.6	.0013015	6.8720	768.33	5.6723	2,545.7	3,665,800	4.0109	1,800.1	2,592,100
2.7	.0013981	7.3819	715.25	5.8905	2,643.6	3,806,800	4.1651	1,869.3	2,691,800
2.8	.0014985	7.9121	667.33	6.1087	2,741.6	3,947,800	4.3195	1,938.6	2,791,500
2.9	.0016020	8.4584	624.22	6.3269	2,839.5	4,088,800	4.4737	2,007.8	2,891,200
3.0	.0017093	9.0251	585.03	6.5450	2,937.4	4,229,800	4.6280	2,077.0	2,990,900
3.1	.0018198	9.6084	549.52	6.7632	3,035.3	4,370,800	4.7822	2,146.2	3,090,600
3.2	.0019343	10.213	516.98	6.9814	3,133.2	4,511,800	4.9365	2,215.5	3,190,300
3.3	.0020510	10.829	487.57	7.1995	3,231.1	4,652,800	5.0907	2,284.7	3,290,000
3.4	.0021718	11.467	460.45	7.4177	3,329.0	4,793,800	5.2451	2,354.0	3,389,700
3.5	.0022946	12.115	435.81	7.6359	3,427.0	4,934,800	5.3993	2,423.2	3,489,400

$d = 20$ *inches* $= 1.6667$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
3·6	.0024239	12·798	412·56	7·8540	3,524·8	5,075,700	5·5535	2,492·4	3,589,100
3·7	.0025566	13·499	391·15	8.0722	3,622·8	5,216,700	5·7078	2,561·7	3,688,800
3·8	.0026913	14·210	371·57	8.2903	3,720·7	5,357,700	5·8621	2,630·9	3,788,400
3·9	.0028305	14·945	353·30	8.5084	3,818·6	5,498,700	6.0163	2,700·1	3,888,100
4·0	.0029731	15·698	336·35	8.7267	3,916·5	5,639,800	6.1706	2,769·4	3,987,900
4·1	.0031188	16·467	320·63	8.9448	4,014·4	5,780,700	6.3248	2,838·6	4,087,500
4·2	.0032680	17·255	306.00	9·1630	4,112·3	5,921,700	6.4792	2,907·8	4,187,300
4·3	.0034185	18·050	292·52	9·3812	4,210·3	6,062,800	6.6335	2,977·1	4,287,000
4·4	.0035740	18·870	279·80	9·5993	4,308·1	6,203,700	6.7877	3,046·3	4,386,600
4·5	.0037326	19·708	267·91	9·8175	4,406·1	6,344,700	6.9419	3,115·5	4,486,300
4·6	.0038945	20·563	256·77	10·036	4,504·0	6,485,700	7·0963	3,184·8	4,586,100
4·7	.0040595	21·434	246·34	10·254	4,601·9	6,626,700	7·2505	3,254·0	4,685,800
4·8	.0042253	22·310	236·67	10·472	4,699·8	6,767,700	7·4047	3,323·2	4,785,400
4·9	.0043966	23·214	227·45	10·690	4,797·8	6,908,800	7·5591	3,392·5	4,885,200
5·0	.0045709	24·134	218·78	10·908	4,895·6	7,049,700	7·7133	3,461·7	4,984,800

5·5	·0054855	28·963	182·30	11·999	5,385·2	7,754,600	8·4846	3,807·9	5,483,300
6·0	·0064746	34·185	154·45	13·090	5,847·8	8,459,600	9·2559	4,154·0	5,981,800
6·5	·0075513	39·870	132·43	14·181	6,364·3	9,164,500	10·027	4,500·2	6,480,200
7·0	·0087030	45·952	114·90	15·272	6,853·9	9,869,600	10·799	4,846·4	6,978,800
7·5	·0099275	52·417	100·73	16·363	7,343·4	10,575,000	11·570	5,192·5	7,477,200
8·0	·011224	59·261	89·096	17·453	7,833·0	11,280,000	12·341	5,538·7	7,975,700
8·5	·012617	66·616	79·259	18·544	8,322·6	11,985,000	13·113	5,884·9	8,474,200
9·0	·014084	74·364	71·002	19·635	8,812·1	12,689,000	13·884	6,231·0	8,972,600
9·5	·015625	82·499	64·000	20·726	9,301·7	13,394,000	14·655	6,577·2	9,471,100
10·0	·017239	91·019	58·010	21·817	9,791·3	14,099,000	15·427	6,923·4	9,969,700
10·5	·018954	100·08	52·758	22·908	10,281	14,804,000	16·198	7,269·6	10,468,000
11·0	·020746	109·54	48·203	23·998	10,770	15,509,000	16·969	7,615·7	10,967,000
11·5	·022613	119·40	44·222	25·089	11,260	16,214,000	17·741	7,962·0	11,465,000
12·0	·024555	129·65	40·725	26·180	11,750	16,919,000	18·512	8,308·1	11,964,000
12·5	·026571	140·29	37·635	27·271	12,239	17,624,000	19·283	8,654·3	12,462,000
13·0	·028660	151·32	34·892	28·362	12,729	18,329,000	20·054	9,000·4	12,960,000
13·5	·030822	162·74	32·444	29·452	13,218	19,034,000	20·826	9,346·5	13,459,000
14·0	·033057	174·54	30·251	30·544	13,708	19,739,000	21·597	9,692·8	13,958,000
14·5	·035421	187·02	28·232	31·634	14,197	20,444,000	22·369	10,039	14,456,000
15·0	·037863	199·92	26·411	32·725	14,687	21,149,000	23·140	10,385	14,954,000

$d = 24$ *inches* $= 2$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.0000017288	.0091281	578,430	.31416	140.99	203,030	.22214	99.697	143,560
.2	.0000068656	.036250	145,650	.62832	281.99	406,060	.44428	199.39	287,120
.3	.000015336	.080971	65,208	.94248	422.98	609,090	.66642	299.09	430,690
.4	.000027065	.14290	36,949	1.2566	563.98	812,120	.88857	398.79	574,250
.5	.000041977	.22164	23,823	1.5708	704.97	1,015,100	1.1107	498.48	717,810
.6	.000060110	.31738	16,636	1.8850	845.96	1,218,200	1.3328	598.18	861,370
.7	.000081210	.42879	12,314	2.1991	986.96	1,421,200	1.5550	697.88	1,004,900
.8	.00010567	.55793	9,463.5	2.5133	1,128.0	1,624,200	1.7771	797.57	1,148,500
.9	.00013298	.70213	7,519.9	2.8274	1,268.9	1,827,300	1.9993	897.26	1,292,100
1.0	.00016325	.86193	6,125.8	3.1416	1,409.9	2,030,300	2.2214	996.97	1,435,600
1.1	.00019639	1.0370	5,091.8	3.4557	1,550.9	2,233,300	2.4435	1,096.7	1,579,200
1.2	.00023283	1.2293	4,295.0	3.7699	1,691.9	2,436,400	2.6657	1,196.4	1,722,700
1.3	.00027168	1.4344	3,680.9	4.0840	1,832.9	2,639,400	2.8878	1,296.0	1,866,300
1.4	.00031326	1.6540	3,192.3	4.3983	1,973.9	2,842,400	3.1100	1,395.8	2,009,900
1.5	.00035820	1.8913	2,791.7	4.7124	2,114.9	3,045,400	3.3321	1,495.4	2,153,400

1.6	.00040597	2.1435	2,463.3	5.0266	2,255.9	3,248,500	3.5543	1,595.1	2,297,000
1.7	.00045650	2.4103	2,190.6	5.3407	2,396.9	3,451,500	3.7764	1,694.8	2,440,600
1.8	.00050976	2.6915	1,961.7	5.6548	2,537.9	3,654,500	3.9985	1,794.5	2,584,100
1.9	.00056461	2.9811	1,771.1	5.9690	2,678.9	3,857,500	4.2207	1,894.2	2,727,700
2.0	.00062313	3.2901	1,604.8	6.2832	2,819.9	4,060,600	4.4428	1,993.9	2,871,200
2.1	.00068495	3.6165	1,460.0	6.5974	2,960.9	4,263,600	4.6650	2,093.6	3,014,800
2.2	.00074908	3.9551	1,335.0	6.9115	3,101.8	4,466,600	4.8871	2,193.3	3,158,300
2.3	.00081587	4.3077	1,225.7	7.2257	3,242.9	4,669,700	5.1093	2,293.0	3,301,900
2.4	.00088477	4.6715	1,130.2	7.5398	3,383.8	4,872,700	5.3314	2,392.7	3,445,500
2.5	.00095616	5.0485	1,045.9	7.8540	3,524.8	5,075,700	5.5535	2,492.4	3,589,100
2.6	.0010310	5.4436	969.93	8.1681	3,665.8	5,278,700	5.7756	2,592.1	3,732,600
2.7	.0011079	5.8495	902.63	8.4822	3,806.8	5,481,800	5.9978	2,691.8	3,876,100
2.8	.0011872	6.2684	842.31	8.7965	3,947.8	5,684,900	6.2200	2,791.5	4,019,800
2.9	.0012683	6.6965	788.46	9.1107	4,088.8	5,887,900	6.4421	2,891.2	4,163,300
3.0	.0013517	7.1367	739.83	9.4248	4,229.8	6,090,900	6.6642	2,990.9	4,306,900
3.1	.0014403	7.6047	694.30	9.7389	4,370.8	6,293,900	6.8864	3,090.6	4,450,400
3.2	.0015315	8.0865	652.94	10.053	4,511.8	6,497,000	7.1085	3,190.3	4,594,000
3.3	.0016253	8.5816	615.26	10.367	4,652.8	6,699,900	7.3306	3,290.0	4,737,500
3.4	.0017218	9.0910	580.79	10.681	4,793.8	6,903,000	7.5528	3,389.7	4,881,100
3.5	.0018207	9.6135	549.22	10.996	4,934.8	7,106,100	7.7750	3,489.4	5,024,700

d = *24 inches* = *2 feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
3.6	.0019222	10.149	520.24	11.310	5,075.7	7,309,000	7.9971	3,589.1	5,168,200
3.7	.0020262	10.698	493.53	11.624	5,216.7	7,512,100	8.2192	3,688.8	5,311,800
3.8	.0021327	11.261	468.89	11.938	5,357.7	7,715,100	8.4413	3,788.4	5,455,300
3.9	.0022417	11.836	446.09	12.252	5,498.7	7,918,100	8.6634	3,888.1	5,598,900
4.0	.0023532	12.425	424.95	12.566	5,639.8	8,121,200	8.8857	3,987.9	5,742,500
4.1	.0024684	13.033	405.13	12.880	5,780.7	8,324,200	9.1077	4,087.5	5,886,000
4.2	.0025862	13.655	386.67	13.195	5,921.7	8,527,300	9.3300	4,187.3	6,029,600
4.3	.0027079	14.298	369.28	13.509	6,062.8	8,730,300	9.5521	4,287.0	6,173,200
4.4	.0028308	14.947	353.26	13.823	6,203.7	8,933,300	9.7742	4,386.6	6,316,700
4.5	.0029562	15.609	338.27	14.137	6,344.7	9,136,300	9.9963	4,486.3	6,460,300
4.6	.0030842	16.284	324.24	14.451	6,485.7	9,339,400	10.219	4,586.1	6,603,900
4.7	.0032146	16.973	311.08	14.766	6,626.7	9,542,500	10.441	4,685.8	6,747,500
4.8	.0033492	17.684	298.58	15.080	6,767.7	9,745,400	10.663	4,785.4	6,891,000
4.9	.0034847	18.399	286.97	15.394	6,908.8	9,948,600	10.885	4,885.2	7,034,600
5.0	.0036225	19.127	276.05	15.708	7,049.7	10,151,000	11.107	4,984.8	7,178,100

5.5	.0043549	22.994	229.63	17.279	7,754.6	11,167,000	12.218	5,483.3	7,895,900
6.0	.0051492	27.188	194.20	18.850	8,459.6	12,182,000	13.328	5,981.8	8,613,700
6.5	.0059971	31.664	166.75	20.420	9,164.5	13,197,000	14.439	6,480.2	9,331,500
7.0	.0069021	36.443	144.88	21.991	9,869.6	14,212,000	15.550	6,978.8	10,049,000
7.5	.0078970	41.696	126.63	23.562	10,575	15,227,000	16.661	7,477.2	10,767,000
8.0	.0089551	47.282	111.67	25.133	11,280	16,242,000	17.771	7,975.7	11,485,000
8.5	.010053	53.081	99.470	26.704	11,985	17,258,000	18.882	8,474.2	12,203,000
9.0	.011208	59.177	89.224	28.274	12,689	18,273,000	19.993	8,972.6	12,921,000
9.5	.012459	65.785	80.260	29.845	13,394	19,288,000	21.103	9,471.1	13,638,000
10.0	.013775	72.729	72.597	31.416	14,099	20,303,000	22.214	9,969.7	14,356,000
10.5	.015161	80.050	65.958	32.987	14,804	21,318,000	23.325	10,468	15,074,000
11.0	.016611	87.704	60.202	34.557	15,509	22,333,000	24.435	10,967	15,792,000
11.5	.018125	95.697	55.173	36.129	16,214	23,349,000	25.546	11,465	16,510,000
12.0	.019701	104.02	50.759	37.699	16,919	24,364,000	26.657	11,964	17,227,000
12.5	.021304	112.49	46.939	39.270	17,624	25,379,000	27.768	12,462	17,945,000
13.0	.022964	121.25	43.547	40.840	18,329	26,394,000	28.878	12,960	18,663,000
13.5	.024679	130.30	40.520	42.411	19,034	27,409,000	29.989	13,459	19,381,000
14.0	.026450	139.66	37.807	43.983	19,739	28,424,000	31.100	13,958	20,099,000
14.5	.028341	149.64	35.285	45.553	20,444	29,439,000	32.211	14,456	20,817,000
15.0	.030294	159.95	33.010	47.124	21,149	30,454,000	33.321	14,954	21,534,000

$d = 30$ *inches* $= 2.5$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q			Q′		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.0000012910	.0068166	774,570	.49087	220.30	317,230	.34710	155.78	224,320
.2	.0000051343	.027109	194,770	.98175	440.61	634,470	.69419	311.55	448,630
.3	.000011485	.060640	87,070	1.4726	660.91	951,700	1.0413	467.33	672,950
.4	.000020298	.10717	49,265	1.9635	881.21	1,268,900	1.3884	623.10	897,260
.5	.000031530	.16648	31,716	2.4544	1,101.5	1,586,200	1.7355	778.88	1,121,600
.6	.000045134	.23830	22,156	2.9452	1,321.8	1,903,400	2.0826	934.65	1,345,900
.7	.000061067	.32243	16,375	3.4361	1,542.1	2,220,600	2.4297	1,090.4	1,570,200
.8	.000079283	.41861	12,613	3.9270	1,762.4	2,537,900	2.7768	1,246.2	1,794,500
.9	.000099738	.52661	10,026	4.4178	1,982.7	2,855,100	3.1238	1,402.0	2,018,800
1.0	.00012239	.64620	8,170.7	4.9087	2,203.0	3,172,300	3.4710	1,557.8	2,243,200
1.1	.00014749	.77872	6,780.3	5.3996	2,423.3	3,489,600	3.8180	1,713.5	2,467,500
1.2	.00017480	.92295	5,720.7	5.8905	2,643.6	3,806,800	4.1651	1,869.3	2,691,800
1.3	.00020431	1.0787	4,894.5	6.3813	2,863.9	4,124,000	4.5122	2,025.1	2,916,100
1.4	.00023598	1.2460	4,237.6	6.8723	3,084.3	4,441,300	4.8594	2,180.9	3,140,400
1.5	.00026977	1.4244	3,706.8	7.3631	3,304.5	4,758,500	5.2064	2,336.6	3,364,700

1.6	.00030567	1.6139	3,271.5	7.8540	3,524.8	5,075,700	5.5535	2,492.4	3,589,100
1.7	.00034292	1.8106	2,916.2	8.3449	3,745.2	5,393,000	5.9007	2,648.2	3,813,400
1.8	.00038282	2.0213	2,612.2	8.8357	3,965.4	5,710,200	6.2477	2,803.9	4,037,700
1.9	.00042475	2.2426	2,354.3	9.3265	4,185.7	6,027,400	6.5948	2,959.7	4,262,000
2.0	.00046766	2.4692	2,138.3	9.8175	4,406.1	6,344,700	6.9419	3,115.5	4,486,300
2.1	.00051340	2.7108	1,947.8	10.308	4,626.4	6,661,900	7.2890	3,271.3	4,710,600
2.2	.00056105	2.9623	1,782.4	10.799	4,846.6	6,979,100	7.6361	3,427.0	4,934,900
2.3	.00061059	3.2239	1,637.8	11.290	5,067.0	7,296,400	7.9833	3,582.9	5,159,300
2.4	.00066196	3.4951	1,510.7	11.781	5,287.3	7,613,600	8.3303	3,738.6	5,383,600
2.5	.00071517	3.7761	1,398.3	12.272	5,507.6	7,930,900	8.6774	3,894.4	5,607,900
2.6	.00077099	4.0708	1,297.0	12.763	5,727.8	8,248,000	9.0244	4,050.1	5,832,200
2.7	.00082827	4.3732	1,207.3	13.253	5,948.1	8,565,200	9.3715	4,205.9	6,056,500
2.8	.00088785	4.6878	1,126.3	13.745	6,168.5	8,882,600	9.7187	4,361.7	6,280,900
2.9	.00094875	5.0093	1,054.0	14.235	6,388.8	9,199,800	10.066	4,517.5	6,505,200
3.0	.0010119	5.3429	988.21	14.726	6,609.1	9,517,000	10.413	4,673.3	6,729,500
3.1	.0010775	5.6893	928.05	15.217	6,829.4	9,834,200	10.760	4,829.0	6,953,800
3.2	.0011450	6.0455	873.37	15.708	7,049.7	10,151,000	11.107	4,984.8	7,178,100
3.3	.0012149	6.4148	823.10	16.199	7,269.9	10,469,000	11.454	5,140.6	7,402,400
3.4	.0012861	6.7906	777.54	16.690	7,490.3	10,786,000	11.801	5,296.4	7,626,800
3.5	.0013591	7.1758	735.80	17.181	7,710.6	11,103,000	12.148	5,452.2	7,851,100

$d = 30$ *inches* $= 2.5$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q: Cubic Feet per Second	Q: Gallons per Minute	Q: Gallons per Day	Q′: Cubic Feet per Second	Q′: Gallons per Minute	Q′: Gallons per Day
3.6	.0014346	7.5746	697.06	17.671	7,930.9	11,420,000	12.495	5,607.9	8,075,300
3.7	.0015120	7.9833	661.38	18.162	8,151.2	11,738,000	12.843	5,763.7	8,299,700
3.8	.0015912	8.4016	628.45	18.653	8,371.4	12,055,000	13.190	5,919.4	8,523,900
3.9	.0016723	8.8296	597.98	19.144	8,591.7	12,372,000	13.537	6,075.2	8,748,200
4.0	.0017552	9.2674	569.73	19.635	8,812.1	12,689,000	13.884	6,231.0	8,972,600
4.1	.0018420	9.7255	542.90	20.126	9,032.3	13,006,000	14.231	6,386.8	9,196,900
4.2	.0019307	10.194	517.94	20.617	9,252.7	13,324,000	14.578	6,542.6	9,421,300
4.3	.0020215	10.673	494.69	21.108	9,473.1	13,641,000	14.925	6,698.4	9,645,600
4.4	.0021141	11.162	473.01	21.598	9,693.3	13,958,000	15.272	6,854.1	9,869,800
4.5	.0022088	11.662	452.73	22.089	9,913.6	14,275,000	15.619	7,009.9	10,094,000
4.6	.0023055	12.173	433.75	22.580	10,134	14,593,000	15.967	7,165.7	10,319,000
4.7	.0024041	12.693	415.96	23.071	10,354	14,910,000	16.314	7,321.5	10,543,000
4.8	.0025046	13.224	399.27	23.562	10,575	15,227,000	16.661	7,477.2	10,767,000
4.9	.0026071	13.765	383.57	24.053	10,795	15,545,000	17.008	7,633.1	10,992,000
5.0	.0027114	14.316	368.81	24.544	11,015	15,862,000	17.355	7,788.8	11,216,000

5·5	·0032582	17·203	306.92	26.998	12,117	17,448,000	19·090	8,567·6	12,337,000
6.0	·0038507	20·332	259·69	29·452	13,218	19,034,000	20·826	9,346·5	13,459,000
6·5	·0045034	23·778	222·05	31·907	14,320	20,620,000	22·561	10,125	14,580,000
7·0	·0052048	27·481	192·13	34·361	15,421	22,206,000	24·297	10,904	15,702,000
7·5	·0059398	31·362	168·36	36·815	16,523	23,792,000	26·032	11,683	16,824,000
8·0	·0067183	35·472	148·85	39·270	17,624	25,379,000	27·768	12,462	17,945,000
8·5	·0075485	39·856	132·48	41·724	18,726	26,965,000	29·503	13,241	19,067,000
9·0	·0084223	44·469	118·73	44·178	19,827	28,551,000	31·238	14,020	20,188,000
9·5	·0093502	49·369	106·95	46·633	20,929	30,137,000	32·974	14,799	21,310,000
10·0	·010323	54·507	96·868	49·087	22,030	31,723,000	34·710	15,578	22,432,000
10·5	·011361	59·985	88·022	51·542	23,132	33,310,000	36·445	16,356	23,553,000
11·0	·012446	65·714	80·347	53·996	24,233	34,896,000	38·180	17,135	24,675,000
11·5	·013579	71·695	73·644	56·451	25,335	36,482,000	39·916	17,914	25,796,000
12·0	·014758	77·922	67·759	58·905	26,436	38,068,000	41·651	18,693	26,918,000
12·5	·015985	84·398	62·560	61·359	27,538	39,654,000	43·387	19,472	28,039,000
13·0	·017257	91·115	57·948	63·813	28,639	41,240,000	45·122	20,251	29,161,000
13·5	·018576	98·080	53·833	66·267	29,741	42,826,000	46·858	21,030	30,282,000
14·0	·019941	105·29	50·148	68·723	30,843	44,413,000	48·594	21,809	31,404,000
14·5	·021352	112·74	46·834	71·177	31,944	45,999,000	50·329	22,588	32,526,000
15·0	·0122808	120·42	43·845	73·631	33,045	47,585,000	52·064	23,366	33,647,000

$d = 36$ *inches* $= 3$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
.1	.0000010054	.0053084	994,650	.70685	317.23	456,810	.49982	224.32	323,010
.2	.0000039967	.021102	250,210	1.4137	634.47	913,630	.99963	448.63	646,030
.3	.0000089366	.047185	111,900	2.1206	951.70	1,370,400	1.4994	672.95	969,040
.4	.000015788	.083359	63,340	2.8274	1,268.9	1,827,300	1.9993	897.26	1,292,100
.5	.000024513	.12943	40,795	3.5343	1,586.2	2,284,100	2.4991	1,121.6	1,615,100
.6	.000035149	.18558	28,451	4.2411	1,903.4	2,740,900	2.9989	1,345.9	1,938,100
.7	.000047639	.25153	20,991	4.9480	2,220.6	3,197,700	3.4987	1,570.2	2,261,100
.8	.000061957	.32713	16,140	5.6548	2,537.9	3,654,500	3.9985	1,794.5	2,584,100
.9	.000077909	.41136	12,835	6.3617	2,855.1	4,111,300	4.4983	2,018.8	2,907,100
1.0	.000095770	.50566	10,442	7.0685	3,172.3	4,568,100	4.9982	2,243.2	3,230,100
1.1	.00011513	.60787	8,686.0	7.7754	3,489.6	5,024,900	5.4979	2,467.5	3,553,100
1.2	.00013642	.72028	7,330.4	8.4822	3,806.8	5,481,800	5.9978	2,691.8	3,876,100
1.3	.00015940	.84161	6,273.6	9.1890	4,124.0	5,938,500	6.4976	2,916.1	4,199,100
1.4	.00018406	.97181	5,433.1	9.8960	4,441.3	6,395,400	6.9975	3,140.4	4,522,200
1.5	.00021035	1.1107	4,753.9	10.603	4,758.5	6,852,200	7.4972	3,364.7	4,845,200

1.6	.00023880	1.2609	4,187.6	11.310	5,075.7	7,309,000	7.9971	3,589.1	5,168,200
1.7	.00026839	1.4171	3,725.9	12.017	5,393.0	7,765,900	8.4969	3,813.4	5,491,200
1.8	.00029954	1.5816	3,338.4	12.723	5,710.2	8,222,600	8.9966	4,037.7	5,814,200
1.9	.00033225	1.7543	3,009.7	13.430	6,027.4	8,679,400	9.4964	4,262.0	6,137,200
2.0	.00036650	1.9351	2,728.5	14.137	6,344.7	9,136,300	9.9963	4,486.3	6,460,300
2.1	.00040224	2.1238	2,486.1	14.844	6,661.9	9,593,100	10.496	4,710.6	6,783,300
2.2	.00043944	2.3202	2,275.6	15.551	6,979.1	10,050,000	10.996	4,934.9	7,106,200
2.3	.00047812	2.5245	2,091.5	16.258	7,296.4	10,507,000	11.496	5,159.3	7,429,300
2.4	.00051820	2.7361	1,929.7	16.964	7,613.6	10,964,000	11.996	5,383.6	7,752,300
2.5	.00055969	2.9552	1,786.7	17.671	7,930.9	11,420,000	12.495	5,607.9	8,075,300
2.6	.00060325	3.1852	1,657.7	18.378	8,248.0	11,877,000	12.995	5,832.2	8,398,300
2.7	.00064790	3.4209	1,543.4	19.085	8,565.2	12,334,000	13.495	6,056.5	8,721,300
2.8	.00069437	3.6662	1,440.2	19.792	8,882.6	12,791,000	13.995	6,280.9	9,044,400
2.9	.00074181	3.9167	1,348.1	20.499	9,199.8	13,248,000	14.495	6,505.2	9,367,400
3.0	.00079104	4.1767	1,264.2	21.206	9,517.0	13,704,000	14.994	6,729.5	9,690,400
3.1	.00084266	4.4492	1,186.7	21.912	9,834.2	14,161,000	15.494	6,953.8	10,013,000
3.2	.00089578	4.7297	1,116.3	22.619	10,151	14,618,000	15.994	7,178.1	10,336,000
3.3	.00095036	5.0179	1,052.2	23.326	10,469	15,075,000	16.494	7,402.4	10,659,000
3.4	.0010065	5.3141	993.57	24.033	10,786	15,532,000	16.994	7,626.8	10,982,000
3.5	.0010640	5.6178	939.85	24.740	11,103	15,989,000	17.494	7,851.1	11,306,000

d = 36 *inches* = 3 *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q′		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
3·6	.0011236	5·9328	889·96	25·447	11,420	16,445,000	17·993	8,075·3	11,628,000
3·7	.0011848	6·2558	844·01	26·154	11,738	16,902,000	18·493	8,299·7	11,951,000
3·8	.0012467	6·5825	802·12	26·860	12,055	17,359,000	18·993	8,523·9	12,274,000
3·9	.0013108	6·9210	762·89	27·567	12,372	17,816,000	19·493	8,748·2	12,597,000
4·0	.0013764	7·2676	726·51	28·274	12,689	18,273,000	19·993	8,972·6	12,921,000
4·1	.0014443	7·6261	692·36	28·981	13,006	18,729,000	20·492	9,196·9	13,243,000
4·2	.0015139	7·9932	660·56	29·688	13,324	19,186,000	20·992	9,421·3	13,567,000
4·3	.0015849	8·3684	630·94	30·395	13,641	19,643,000	21·492	9,645·6	13,890,000
4·4	.0016574	8·7512	603·34	31·101	13,958	20,100,000	21·992	9,869·8	14,212,000
4·5	.0017315	9·1424	577·52	31·808	14,275	20,557,000	22·492	10,094	14,536,000
4·6	.0018072	9·5420	553·34	32·515	14,593	21,014,000	22·992	10,319	14,859,000
4·7	.0018843	9·9492	530·69	33·222	14,910	21,470,000	23·491	10,543	15,182,000
4·8	.0019630	10·364	509·44	33·929	15,227	21,927,000	23·991	10,767	15,505,000
4·9	.0020431	10·788	489·44	34·636	15,545	22,384,000	24·491	10,992	15,828,000
5·0	.0021248	11·219	470·64	35·343	15,862	22,841,000	24·991	11,216	16,151,000

5·5	·0025553	13·492	391·35	38·877	17,448	25,125,000	27·490	12,337	17,766,000
6·0	·0030224	15·958	330·86	42·411	19,034	27,409,000	29·989	13,459	19,381,000
6·5	·0035296	18·636	283·32	45·945	20,620	29,693,000	32·488	14,580	20,996,000
7·0	·0040731	21·506	245·51	49·480	22,206	31,977,000	34·987	15,702	22,611,000
7·5	·0046583	24·596	214·67	53·014	23,792	34,261,000	37·486	16,824	24,226,000
8·0	·0052802	27·879	189·39	56·548	25,379	36,545,000	39·985	17,945	25,841,000
8·5	·0059384	31·355	168·40	60·083	26,965	38,829,000	42·484	19,067	27,456,000
9·0	·0066324	35·019	150·78	63·617	28,551	41,113,000	44·983	20,188	29,071,000
9·5	·0073616	38·869	135·84	67·151	30,137	43,397,000	47·482	21,310	30,686,000
10·0	·0081261	42·905	123·06	70·685	31,723	45,681,000	49·982	22,432	32,301,000
10·5	·0089417	47·212	111·84	74·220	33,310	47,966,000	52·481	23,553	33,916,000
11·0	·0097947	51·715	102·10	77·754	34,896	50,249,000	54·979	24,675	35,531,000
11·5	·010685	56·417	93·588	81·289	36,482	52,534,000	57·479	25,796	37,147,000
12·0	·011612	61·310	86·119	84·822	38,068	54,818,000	59·978	26,918	38,761,000
12·5	·012575	66·397	79·521	88·357	39,654	57,102,000	62·477	28,039	40,377,000
13·0	·013575	71·675	73·665	91·890	41,240	59,385,000	64·976	29,161	41,991,000
13·5	·014611	77·145	68·442	95·425	42,826	61,669,000	67·475	30,282	43,606,000
14·0	·015683	82·808	63·762	98·960	44,413	63,954,000	69·975	31,404	45,222,000
14·5	·016791	88·654	59·557	102·49	45,999	66,238,000	72·474	32,526	46,837,000
15·0	·017934	94·689	55·761	106·03	47,585	68,522,000	74·972	33,647	48,452,000

$d = 42$ *inches* $= 3.5$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q′		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.00000078179	.0041278	1,279,100	.96212	431.80	621,780	.68031	305.32	439,660
.2	.0000031058	.016399	321,970	1.9242	863.59	1,243,600	1.3606	610.65	879,330
.3	.0000069402	.036644	144,090	2.8864	1,295.4	1,865,300	2.0409	915.97	1,319,000
.4	.000012253	.064695	81,613	3.8485	1,727.2	2,487,100	2.7213	1,221.3	1,758,700
.5	.000019012	.10038	52,599	4.8106	2,159.0	3,108,900	3.4016	1,526.6	2,198,300
.6	.000027249	.14387	36,699	5.7727	2,590.8	3,730,700	4.0819	1,831.9	2,638,000
.7	.000036916	.19491	27,089	6.7349	3,022.6	4,352,500	4.7622	2,137.3	3,077,700
.8	.000048045	.25368	20,814	7.6970	3,454.4	4,974,300	5.4425	2,442.6	3,517,300
.9	.000060590	.31991	16,504	8.6590	3,886.2	5,596,000	6.1228	2,747.9	3,956,900
1.0	.000074537	.39355	13,416	9.6212	4,318.0	6,217,800	6.8031	3,053.2	4,396,600
1.1	.000089867	.47449	11,128	10.583	4,749.7	6,839,600	7.4834	3,358.5	4,836,300
1.2	.00010656	.56265	9,384.0	11.545	5,181.6	7,461,400	8.1638	3,663.9	5,275,900
1.3	.00012461	.65796	8,024.7	12.507	5,613.3	8,083,100	8.8440	3,969.2	5,715,600
1.4	.00014418	.76125	6,935.9	13.470	6,045.2	8,705,000	9.5245	4,274.5	6,155,300
1.5	.00016491	.87070	6,064.0	14.432	6,476.9	9,326,700	10.205	4,579.8	6,594,900

1.6	.00018718	.98828	5,342.6	15.394	6,908.8	9,948,600	10.885	4,885.2	7,034,600
1.7	.00021079	1.1130	4,744.1	16.356	7,340.6	10,570,000	11.565	5,190.5	7,474,300
1.8	.00023574	1.2447	4,242.0	17.318	7,772.3	11,192,000	12.246	5,495.8	7,913,900
1.9	.00026202	1.3834	3,816.5	18.280	8,204.1	11,814,000	12.926	5,801.1	8,353,500
2.0	.00028962	1.5292	3,452.8	19.242	8,635.9	12,436,000	13.606	6,106.5	8,793,300
2.1	.00031852	1.6818	3,139.5	20.205	9,067.8	13,057,000	14.287	6,411.8	9,232,900
2.2	.00034871	1.8412	2,867.7	21.167	9,499.5	13,679,000	14.967	6,717.1	9,672,500
2.3	.00038021	2.0075	2,630.1	22.129	9,931.4	14,301,000	15.647	7,022.5	10,112,000
2.4	.00041270	2.1790	2,423.1	23.091	10,363	14,923,000	16.328	7,327.7	10,552,000
2.5	.00044643	2.3571	2,240.0	24.053	10,795	15,545,000	17.008	7,633.1	10,992,000
2.6	.00048164	2.5430	2,076.3	25.015	11,227	16,166,000	17.688	7,938.3	11,431,000
2.7	.00051811	2.7356	1,930.1	25.977	11,658	16,788,000	18.368	8,243.7	11,871,000
2.8	.00055616	2.9365	1,798.0	26.940	12,090	17,410,000	19.049	8,549.1	12,311,000
2.9	.00059548	3.1441	1,679.3	27.902	12,522	18,032,000	19.729	8,854.4	12,750,000
3.0	.00063644	3.3604	1,571.2	28.864	12,954	18,653,000	20.409	9,159.7	13,190,000
3.1	.00067830	3.5814	1,474.3	29.826	13,386	19,275,000	21.090	9,465.0	13,630,000
3.2	.00072141	3.8090	1,386.2	30.788	13,818	19,897,000	21.770	9,770.3	14,069,000
3.3	.00076526	4.0405	1,306.7	31.750	14,249	20,519,000	22.450	10,076	14,509,000
3.4	.00081081	4.2810	1,233.3	32.712	14,681	21,141,000	23.131	10,381	14,949,000
3.5	.00085759	4.5280	1,166.1	33.674	15,113	21,763,000	23.811	10,686	15,388,000

$d = 42$ *inches* $= 3.5$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
3·6	.00090611	4·7842	1,103·6	34·636	15,545	22,384,000	24·491	10,992	15,828,000
3·7	.00095594	5·0437	1,046.1	35·598	15,976	23,006,000	25·172	11,297	16,267,000
3·8	.0010070	5·3170	993·02	36.560	16,408	23,628,000	25·852	11,602	16,707,000
3·9	.0010594	5·5935	943·95	37·522	16,840	24,249,000	26.532	11,907	17,147,000
4·0	.0011130	5·8765	898.48	38·485	17,272	24,871,000	27·213	12,213	17,587,000
4·1	.0011686	6.1699	855·76	39·447	17,704	25,493,000	27·893	12,518	18,026,000
4·2	.0012247	6.4665	816.51	40·409	18,136	26,115,000	28·573	12,823	18,466,000
4·3	.0012829	6·7738	779·47	41·371	18,567	26,737,000	29·254	13,129	18,906,000
4·4	.0013415	7·0832	745·42	42·333	18,999	27,358,000	29·934	13,434	19,345,000
4·5	.0014023	7·4042	713·10	43·295	19,431	27,980,000	30.614	13,739	19,785,000
4·6	.0014644	7·7321	682.86	44·258	19,863	28,602,000	31·295	14,045	20,225,000
4·7	.0015268	8.0616	654·95	45·220	20,295	29,224,000	31·975	14,350	20,664,000
4·8	.0015914	8.4027	628.36	46.182	20,726	29,846,000	32·655	14,655	21,104,000
4·9	.0016563	8·7454	603·74	47·144	21,158	30,468,000	33·336	14,961	21,544,000
5·0	.0017235	9.1000	580.22	48.106	21,590	31,089,000	34·016	15,266	21,983,000

5·5	.0020747	10.954	482.00	52.916	23,749	34,198,000	37.417	16,793	24,181,000
6.0	.0024562	12.969	407.13	57.727	25,908	37,307,000	40.819	18,319	26,380,000
6.5	.0028695	15.151	348.49	62.537	28,067	40,416,000	44.220	19,846	28,578,000
7.0	.0033128	17.491	301.86	67.349	30,226	43,525,000	47.622	21,373	30,777,000
7.5	.0037879	20.000	264.00	72.159	32,385	46,634,000	51.023	22,899	32,975,000
8.0	.0042928	22.666	232.95	76.970	34,544	49,743,000	54.425	24,426	35,173,000
8.5	.0048365	25.536	206.76	81.781	36,703	52,852,000	57.827	25,953	37,371,000
9.0	.0054114	28.572	184.79	86.590	38,862	55,960,000	61.228	27,479	39,569,000
9.5	.0060092	31.729	166.41	91.401	41,020	59,069,000	64.629	29,005	41,768,000
10.0	.0066364	35.040	150.69	96.212	43,180	62,178,000	68.031	30,532	43,966,000
10.5	.0073020	38.554	136.95	101.02	45,339	65,287,000	71.433	32,059	46,165,000
11.0	.0079976	42.227	125.04	105.83	47,497	68,396,000	74.834	33,585	48,363,000
11.5	.0087239	46.062	114.63	110.64	49,657	71,506,000	78.237	35,112	50,562,000
12.0	.0094796	50.052	105.49	115.45	51,816	74,614,000	81.638	36,639	52,759,000
12.5	.010265	54.200	97.416	120.27	53,975	77,723,000	85.039	38,165	54,958,000
13.0	.011088	58.542	90.190	125.07	56,133	80,831,000	88.440	39,692	57,156,000
13.5	.011933	63.004	83.803	129.89	58,292	83,940,000	91.842	41,218	59,354,000
14.0	.012816	67.667	78.028	134.70	60,452	87,050,000	95.245	42,745	61,553,000
14.5	.013720	72.439	72.889	139.51	62,611	90,159,000	98.646	44,272	63,751,000
15.0	.014652	77.361	68.251	144.32	64,769	93,267,000	102.05	45,798	65,949,000

$d = 48\ inches = 4\ feet$

v	$s = \frac{h}{l}$	$s_m = 5,280\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.00000062810	.0033163	1,592,100	1.2566	563.98	812,120	.88857	398.79	574,250
.2	.0000025062	.013233	399,010	2.5133	1,128.0	1,624,200	1.7771	797.57	1,148,500
.3	.0000056250	.029700	177,780	3.7699	1,691.9	2,436,400	2.6657	1,196.4	1,722,700
.4	.0000099749	.052667	100,250	5.0266	2,255.9	3,248,500	3.5543	1,595.1	2,297,000
.5	.000015547	.082088	64,321	6.2832	2,819.9	4,060,600	4.4428	1,993.9	2,871,200
.6	.000022332	.11791	44,780	7.5398	3,383.8	4,872,700	5.3314	2,392.7	3,445,500
.7	.000030320	.16009	32,981	8.7965	3,947.8	5,684,900	6.2200	2,791.5	4,019,800
.8	.000039502	.20857	25,315	10.053	4,501.8	6,497,000	7.1085	3,190.3	4,594,000
.9	.000049869	.26331	20,053	11.310	5,075.7	7,309,000	7.9971	3,589.1	5,168,200
1.0	.000061412	.32425	16,284	12.566	5,63[illegible].8	8,121,200	8.8857	3,987.9	5,742,500
1.1	.000074119	.39135	13,492	13.823	6,203.7	8,933,300	9.7742	4,386.6	6,316,700
1.2	.000087983	.46455	11,366	15.080	6,767.7	9,745,400	10.663	4,785.4	6,891,000
1.3	.00010300	.54381	9,709.1	16.336	7,331.6	10,557,000	11.551	5,184.2	7,465,200
1.4	.00011915	.62910	8,392.9	17.5[illegible]3	7,895.7	11,370,000	12.440	5,583.0	8,039,500
1.5	.00013642	.72031	7,330.1	18.850	8,459.6	12,182,000	13.328	5,981.8	8,613,700

1·6	·00015443	·81536	6,475·6	20·106	9,023·6	12,994,000	14·217	6,380·6	9,188,000
1·7	·00017388	·91810	5,751·0	21·363	9,587·6	13,806,000	15·106	6,779·4	9,762,300
1·8	·00019444	1·0266	5,143·0	22·619	10,151	14,618,000	15·994	7,178·1	10,336,000
1·9	·00021608	1·1409	4,628·0	23·876	10,715	15,430,000	16·883	7,576·9	10,911,000
2·0	·00023880	1·2609	4,187·6	25·133	11,280	16,242,000	17·771	7,975·7	11,485,000
2·1	·00026277	1·3874	3,805·7	26·389	11,843	17,055,000	18·660	8,374·5	12,059,000
2·2	·00028782	1·5197	3,474·4	27·646	12,407	17,867,000	19·548	8,773·2	12,633,000
2·3	·00031376	1·6566	3,187·1	28·903	12,971	18,679,000	20·437	9,172·1	13,208,000
2·4	·00034097	1·8003	2,932·9	30·159	13,535	19,491,000	21·326	9,570·8	13,782,000
2·5	·00036924	1·9496	2,708·3	31·416	14,099	20,303,000	22·214	9,969·7	14,356,000
2·6	·00039858	2·1045	2,508·9	32·672	14,663	21,115,000	23·103	10,368	14,930,000
2·7	·00042898	2·2650	2,331·1	33·929	15,227	21,927,000	23·991	10,767	15,505,000
2·8	·00046014	2·4295	2,173·3	35·186	15,791	22,739,000	24·880	11,166	16,079,000
2·9	·00049261	2·6009	2,030·0	36·443	16,355	23,552,000	25·769	11,565	16,653,000
3·0	·00052611	2·7779	1,900·7	37·699	16,919	24,364,000	26·657	11,964	17,227,000
3·1	·00056102	2·9622	1,782·5	38·956	17,483	25,176,000	27·545	12,362	17,802,000
3·2	·00059661	3·1501	1,676·1	40·212	18,047	25,988,000	28·434	12,761	18,376,000
3·3	·00063362	3·3455	1,578·2	41·469	18,611	26,800,000	29·322	13,160	18,950,000
3·4	·00067127	3·5443	1,489·7	42·726	19,175	27,612,000	30·211	13,559	19,525,000
3·5	·00071040	3·7509	1,407·7	43·983	19,739	28,424,000	31·100	13,958	20,099,000

$d = 48 \text{ inches} = 4 \text{ feet}$

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q: Cubic Feet per Second	Q: Gallons per Minute	Q: Gallons per Day	Q': Cubic Feet per Second	Q': Gallons per Minute	Q': Gallons per Day
3.6	.00075055	3.9629	1,332.4	45.239	20,303	29,236,000	31.988	14,356	20,673,000
3.7	.00079123	4.1776	1,263.9	46.495	20,867	30,048,000	32.877	14,755	21,247,000
3.8	.00083345	4.4006	1,199.8	47.752	21,431	30,860,000	33.765	15,154	21,821,000
3.9	.00087611	4.6258	1,141.4	49.008	21,995	31,672,000	34.654	15,552	22,395,000
4.0	.00092039	4.8596	1,086.5	50.266	22,559	32,485,000	35.543	15,951	22,970,000
4.1	.00096565	5.0986	1,035.6	51.522	23,123	33,297,000	36.431	16,350	23,544,000
4.2	.0010127	5.3469	987.48	52.779	23,687	34,109,000	37.320	16,749	24,118,000
4.3	.0010600	5.5969	943.37	54.036	24,251	34,921,000	38.209	17,148	24,693,000
4.4	.0011091	5.8563	901.59	55.292	24,815	35,733,000	39.097	17,546	25,267,000
4.5	.0011586	6.1172	863.14	56.548	25,379	36,545,000	39.985	17,945	25,841,000
4.6	.0012090	6.3835	827.12	57.806	25,943	37,358,000	40.874	18,344	26,416,000
4.7	.0012613	6.6595	792.85	59.062	26,507	38,170,000	41.763	18,743	26,990,000
4.8	.0013137	6.9363	761.20	60.318	27,071	38,982,000	42.651	19,142	27,564,000
4.9	.0013681	7.2235	730.94	61.576	27,635	39,794,000	43.540	19,541	28,138,000
5.0	.0014226	7.5110	702.96	62.832	28,199	40,606,000	44.428	19,939	28,712,000

5·5	·0017142	9·0511	583·35	69·115	31,018	44,666,000	48·871	21,933	31,583,000
6·0	·0020317	10·727	492·20	75·398	33,838	48,727,000	53·314	23,927	34,455,000
6·5	·0023778	12·555	420·55	81·681	36,658	52,787,000	57·756	25,921	37,325,000
7·0	·0027502	14·521	363·61	87·965	39,478	56,849,000	62·200	27,915	40,198,000
7·5	·0031439	16·600	318·08	94·248	42,298	60,909,000	66·642	29,909	43,069,000
8·0	·0035621	18·808	280·73	100·53	45,118	64,970,000	71·085	31,903	45,940,000
8·5	·0040045	21·144	249·72	106·81	47,938	69,030,000	75·528	33,897	48,811,000
9·0	·0044705	23·604	223·69	113·10	50,757	73,090,000	79·971	35,891	51,682,000
9·5	·0049670	26·225	201·33	119·38	53,577	77,151,000	84·413	37,884	54,553,000
10·0	·0054881	28·977	182·21	125·66	56,398	81,212,000	88·857	39,879	57,425,000
10·5	·0060421	31·902	165·50	131·95	59,217	85,273,000	93·300	41,873	60,296,000
11·0	·0066217	34·962	151·02	138·23	62,037	89,333,000	97·742	43,866	63,167,000
11·5	·0072274	38·160	138·36	144·51	64,857	93,394,000	102·19	45,861	66,039,000
12·0	·0078581	41·491	127·26	150·80	67,677	97,454,000	106·63	47,854	68,910,000
12·5	·0085206	44·988	117·36	157·08	70,497	101,510,000	111·07	49,848	71,781,000
13·0	·0092092	48·624	108·59	163·36	73,316	105,570,000	115·51	51,842	74,652,000
13·5	·0099241	52·399	100·77	169·64	76,136	109,640,000	119·96	53,836	77,523,000
14·0	·010665	56·313	93·761	175·93	78,957	113,700,000	124·40	55,830	80,395,000
14·5	·011416	60·278	87·593	182·21	81,777	117,760,000	128·84	57,824	83,266,000
15·0	·012191	64·367	82·029	188·50	84,596	121,820,000	133·28	59,818	86,137,000

$d = 54 \text{ inches} = 4.5 \text{ feet}$

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
.1	.00000052791	.0027873	1,894,300	1.5904	713.77	1,027,800	1.1246	504.71	726,780
.2	.0000021061	.011120	474,810	3.1808	1,427.5	2,055,700	2.2492	1,009.4	1,453,600
.3	.0000047263	.024955	211,580	4.7712	2,141.3	3,083,500	3.3737	1,514.1	2,180,300
.4	.0000083803	.044248	119,330	6.3617	2,855.1	4,111,300	4.4983	2,018.8	2,907,100
.5	.000013060	.068954	76,572	7.9521	3,568.9	5,139,100	5.6229	2,523.5	3,633,900
.6	.000018756	.099031	53,316	9.5425	4,282.6	6,166,900	6.7475	3,028.2	4,360,600
.7	.000025462	.13444	39,274	11.133	4,996.4	7,194,800	7.8721	3,533.0	5,087,400
.8	.000033167	.17512	30,150	12.723	5,710.2	8 222,600	8.9966	4,037.7	5,814,200
.9	.000041865	.22104	23,886	14.314	6,423.9	9,250,400	10.121	4,542.3	6,540,900
1.0	.000051548	.27217	19,399	15.904	7,137.7	10,278,000	11.246	5,047.1	7,267,800
1.1	.000062204	.32844	16,076	17.494	7,851.5	11,306,000	12.370	5,551.8	7,994,500
1.2	.000073830	.38982	13,545	19.085	8,565.2	12,334,000	13.495	6,056.5	8,721,300
1.3	.000086413	.45626	11,572	20.675	9,279.0	13,362,000	14.619	6,561.2	9,448,000
1.4	.000099952	.52774	10,005	22.266	9,992.9	14,390,000	15.744	7,065.9	10,175,000
1.5	.00011443	.60417	8,739.2	23.856	10,707	15,417,000	16.869	7,570.6	10,902,000

1.6	.00013019	.68742	7,680.9	25.447	11,420	16,445,000	17.993	8,075.3	11,628,000
1.7	.00014658	.77393	6,822.3	27.037	12,134	17,473,000	19.118	8,580.1	12,355,000
1.8	.00016388	.86527	6,102.1	28.627	12,848	18,501,000	20.242	9,084.7	13,082,000
1.9	.00018209	.96144	5,491.7	30.218	13,562	19,529,000	21.367	9,589.4	13,809,000
2.0	.00020121	1.0624	4,969.8	31.808	14,275	20,557,000	22.492	10,094.0	14,536,000
2.1	.00022139	1.1689	4,517.0	33.399	14,989	21,584,000	23.616	10,599	15,262,000
2.2	.00024247	1.2802	4,124.3	34.989	15,703	22,612,000	24.741	11,104	15,989,000
2.3	.00026428	1.3954	3,783.8	36.580	16,417	23,640,000	25.865	11,608	16,716,000
2.4	.00028716	1.5162	3,482.3	38.170	17,130	24,668,000	26.990	12,113	17,443,000
2.5	.00031094	1.6418	3,216.0	39.760	17,844	25,696,000	28.114	12,618	18,169,000
2.6	.00033561	1.7720	2,979.6	41.350	18,558	26,723,000	29.239	13,122	18,896,000
2.7	.00036117	1.9070	2,768.8	42.941	19,272	27,751,000	30.363	13,627	19,623,000
2.8	.00038761	2.0466	2,579.9	44.532	19,986	28,779,000	31.488	14,132	20,350,000
2.9	.00041551	2.1939	2,406.7	46.122	20,700	29,807,000	32.613	14,637	21,077,000
3.0	.00044403	2.3444	2,252.1	47.712	21,413	30,835,000	33.737	15,141	21,803,000
3.1	.00047312	2.4980	2,113.6	49.303	22,127	31,863,000	34.862	15,646	22,530,000
3.2	.00050308	2.6563	1,987.7	50.893	22,841	32,890,000	35.987	16,151	23,257,000
3.3	.00053351	2.8169	1,874.4	52.483	23,554	33,918,000	37.111	16,655	23,983,000
3.4	.00056515	2.9839	1,769.5	54.074	24,268	34,946,000	38.236	17,160	24,710,000
3.5	.00059760	3.1553	1,673.4	55.665	24,982	35,974,000	39.360	17,665	25,437,000

d = 54 inches = 4.5 feet

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q: Cubic Feet per Second	Q: Gallons per Minute	Q: Gallons per Day	Q′: Cubic Feet per Second	Q′: Gallons per Minute	Q′: Gallons per Day
3·6	.00063134	3·3334	1,583·9	57·255	25,696	37,002,000	40·485	18,169	26,164,000
3·7	.00066547	3·5137	1,502·7	58·845	26,409	38,029,000	41·609	18,674	26,891,000
3·8	.00070094	3·7009	1,426·7	60·435	27,123	39,057,000	42·734	19,179	27,617,000
3·9	.00073673	3·8899	1,357·3	62·025	27,837	40,085,000	43·858	19,683	28,344,000
4·0	.00077391	4·0862	1,292·1	63·617	28,551	41,113,000	44·983	20,188	29,071,000
4·1	.00081191	4·2869	1,231·7	65·206	29,264	42,141,000	46·107	20,693	29,798,000
4·2	.00085080	4·4922	1,175·4	66·797	29,979	43,159,000	47·232	21,198	30,525,000
4·3	.00089051	4·7019	1,122·9	68·388	30,692	44,197,000	48·357	21,703	31,251,000
4·4	.00093107	4·9160	1,074·0	69·978	31,406	45,224,000	49·481	22,207	31,978,000
4·5	.00097246	5·1345	1,028·3	71·568	32,120	46,252,000	50·606	22,712	32,705,000
4·6	.0010147	5·3577	985·49	73·159	32,834	47,280,000	51·731	23,217	33,432,000
4·7	.0010578	5·5851	945·37	74·750	33,547	48,308,000	52·855	23,721	34,159,000
4·8	.0011017	5·8169	907·70	76·340	34,261	49,336,000	53·980	24,226	34,885,000
4·9	.0011464	6·0531	872·27	77·931	34,975	50,364,000	55·105	24,731	35,612,000
5·0	.0011920	6·2935	838·96	79·521	35,689	51,391,000	56·229	25,235	36,339,000

5·5	·0014318	7·5598	698·43	87·472	39,257	56,530,000	61·851	27,759	39,972,000
6·0	·0016915	8·9312	591·18	95·425	42,826	61,669,000	67·475	30,282	43,606,000
6·5	·0019793	10·451	505·22	103·38	46,395	66,808,000	73·097	32,806	47,240,000
7·0	·0022889	12·085	436·90	111·33	49,964	71,948,000	78·721	35,330	50,874,000
7·5	·0026158	13·811	382·29	119·28	53,533	77,087,000	84·343	37,853	54,508,000
8·0	·0029629	15·644	337·50	127·23	57,102	82,226,000	89·966	40,377	58,142,000
8·5	·0033300	17·582	300·30	135·19	60,671	87,366,000	95·589	42,900	61,776,000
9·0	·0037164	19·622	269·08	143·14	64,239	92,504,000	101·21	45,423	65,409,000
9·5	·0041345	21·830	241·87	151·09	67,808	97,643,000	106·83	47,947	69,043,000
10·0	·0045744	24·152	218·61	159·04	71,377	102,780,000	112·46	50,471	72,678,000
10·5	·0050394	26·608	198·44	166·99	74,946	107,920,000	118·08	52,994	76,312,000
11·0	·0055265	29·180	180·95	174·94	78,515	113,060,000	123·70	55,518	79,945,000
11·5	·0060359	31·869	165·68	182·90	82,084	118,200,000	129·33	58,042	83,580,000
12·0	·0065670	34·674	152·28	190·85	85,652	123,340,000	134·95	60,565	87,213,000
12·5	·0071203	37·595	140·44	198·80	89,222	128,480,000	140·57	63,088	90,847,000
13·0	·0076956	40·632	129·95	206·75	92,790	133,620,000	146·19	65,612	94,480,000
13·5	·0082926	43·784	120·59	214·70	96,358	138,760,000	151·82	68,135	98,114,000
14·0	·0089117	47·053	112·21	222·66	99,929	143,900,000	157·44	70,659	101,750,000
14·5	·0095523	50·436	104·69	230·61	103,500	149,040,000	163·06	73,183	105,380,000
15·0	·010215	53·932	97·899	238·56	107,070	154,170,000	168·69	75,706	109,020,000

$d = 60$ *inches* $= 5$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\,\frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
.1	.00000045025	.0023773	2,221,000	1.9635	881.21	1,268,900	1.3884	623.10	897,260
.2	.0000017960	.0094829	556,790	3.9270	1,762.4	2,537,900	2.7768	1,246.2	1,794,500
.3	.0000040298	.021277	248,150	5.8905	2,643.6	3,806,800	4.1651	1,869.3	2,691,800
.4	.0000071441	.037721	139,970	7.8540	3,524.8	5,075,700	5.5535	2,492.4	3,589,100
.5	.000011132	.058775	89,834	9.8175	4,406.1	6,344,700	6.9419	3,115.5	4,486,300
.6	.000015985	.084400	62,559	11.781	5,287.3	7,613,600	8.3303	3,738.6	5,383,600
.7	.000021697	.11456	46,090	13.745	6,168.5	8,882,600	9.7187	4,361.7	6,280,900
.8	.000028259	.14920	35,388	15.708	7,049.7	10,151,000	11.107	4,984.8	7,178,100
.9	.000035663	.18830	28,040	17.671	7,930.9	11,420,000	12.495	5,607.9	8,075,300
1.0	.000043905	.23181	22,777	19.635	8,812.1	12,689,000	13.884	6,231.0	8,972,600
1.1	.000052974	.27970	18,877	21.598	9,693.3	13,958,000	15.272	6,854.1	9,869,800
1.2	.000062865	.33193	15,907	23.562	10,575	15,227,000	16.661	7,477.2	10,767,000
1.3	.000073568	.38844	13,593	25.525	11,456	16,496,000	18.049	8,100.3	11,664,000
1.4	.000085324	.45051	11,720	27.489	12,337	17,765,000	19.437	8,723.5	12,562,000
1.5	.000097667	.51568	10,239	29.452	13,218	19,034,000	20.826	9,346.5	13,459,000

1.6	.00011081	.58505	9,024.9	31.416	14,099	20,303,000	22.214	9,969.7	14,356,000
1.7	.00012473	.65857	8,017.3	33.380	14,981	21,572,000	23.603	10,593	15,254,000
1.8	.00013983	.73831	7,151.4	35.343	15,862	22,841,000	24.991	11,216	16,151,000
1.9	.00015535	.82026	6,436.9	37.306	16,743	24,110,000	26.379	11,839	17,048,000
2.0	.00017214	.90889	5,809.2	39.270	17,624	25,379,000	27.768	12,462	17,945,000
2.1	.00018937	.99986	5,280.7	41.233	18,505	26,648,000	29.156	13,085	18,843,000
2.2	.00020738	1.0950	4,822.0	43.197	19,387	27,916,000	30.544	13,708	19,740,000
2.3	.00022601	1.1933	4,424.6	45.161	20,268	29,186,000	31.933	14,331	20,637,000
2.4	.00024555	1.2965	4,072.5	47.124	21,149	30,454,000	33.321	14,954	21,534,000
2.5	.00026586	1.4037	3,761.4	49.087	22,030	31,723,000	34.710	15,578	22,432,000
2.6	.00028712	1.5160	3,482.8	51.051	22,911	32,992,000	36.098	16,201	23,329,000
2.7	.00030941	1.6337	3,232.0	53.014	23,792	34,261,000	37.486	16,824	24,226,000
2.8	.00033203	1.7531	3,011.8	54.978	24,674	35,530,000	38.875	17,447	25,123,000
2.9	.00035539	1.8764	2,813.8	56.942	25,555	36,799,000	40.263	18,070	26,021,000
3.0	.00037947	2.0036	2,635.2	58.905	26,436	38,068,000	41.651	18,693	26,918,000
3.1	.00040459	2.1362	2,471.6	60.868	27,317	39,337,000	43.040	19,316	27,815,000
3.2	.00043080	2.2746	2,321.2	62.832	28,199	40,606,000	44.428	19,939	28,712,000
3.3	.00045747	2.4154	2,185.9	64.795	29,080	41,875,000	45.816	20,562	29,609,000
3.4	.00048526	2.5621	2,060.8	66.759	29,961	43,144,000	47.205	21,186	30,507,000
3.5	.00051346	2.7111	1,947.6	68.723	30,843	44,413,000	48.594	21,809	31,404,000

$d = 60$ *inches* $= 5$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280 \frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
3.6	.00054200	2.8617	1,845.0	70.685	31,723	45,681,000	49.982	22,432	32,301,000
3.7	.00057126	3.0162	1,750.5	72.649	32,605	46,950,000	51.370	23,055	33,199,000
3.8	.00060076	3.1720	1,664.6	74.612	33,486	48,219,000	52.758	23,678	34,096,000
3.9	.00063136	3.3336	1,583.9	76.576	34,367	49,488,000	54.146	24,301	34,993,000
4.0	.00066267	3.4989	1,509.0	78.540	35,248	50,757,000	55.535	24,924	35,891,000
4.1	.00069517	3.6705	1,438.5	80.503	36,129	52,026,000	56.923	25,547	36,787,000
4.2	.00072895	3.8488	1,371.8	82.467	37,011	53,295,000	58.312	26,170	37,685,000
4.3	.00076294	4.0283	1,310.7	84.431	37,892	54,564,000	59.701	26,794	38,582,000
4.4	.00079822	4.2145	1,252.8	86.393	38,773	55,833,000	61.089	27,416	39,479,000
4.5	.00083366	4.4017	1,199.5	88.357	39,654	57,102,000	62.477	28,039	40,377,000
4.6	.00086982	4.5926	1,149.7	90.321	40,536	58,371,000	63.866	28,663	41,274,000
4.7	.00090736	4.7908	1,102.1	92.285	41,417	59,640,000	65.254	29,286	42,172,000
4.8	.00094493	4.9892	1,058.3	94.248	42,298	60,909,000	66.642	29,909	43,069,000
4.9	.00098401	5.1955	1,016.2	96.212	43,180	62,178,000	68.031	30,532	43,966,000
5.0	.0010230	5.4014	977.51	98.175	44,061	63,447,000	69.419	31,155	44,863,000

5·5	·0012284	6·4859	814·07	107·99	48,466	69,791,000	76·361	34,270	49,349,000
6·0	·0014507	7·6598	689·30	117·81	52,873	76,136,000	83·303	37,386	53,836,000
6·5	·0016973	8·9617	589·17	127·63	57,278	82,480,000	90·244	40,501	58,322,000
7·0	·0019625	10·362	509·57	137·45	61,685	88,826,000	97·187	43,617	62,809,000
7·5	·0022458	11·858	445·28	147·26	66,091	95,170,000	104·13	46,733	67,295,000
8·0	·0025472	13·449	392·58	157·08	70,497	101,510,000	111·07	49,848	71,781,000
8·5	·0028666	15·136	348·84	166·90	74,903	107,860,000	118·01	52,964	76,268,000
9·0	·0032037	16·915	312·14	176·71	79,309	114,200,000	124·95	56,079	80,753,000
9·5	·0035583	18·788	281·03	186·53	83,714	120,550,000	131·90	59,194	85,239,000
10·0	·0039303	20·752	254·43	196·35	88,121	126,890,000	138·84	62,310	89,726,000
10·5	·0043229	22·825	231·32	206·17	92,527	133,240,000	145·78	65,426	94,213,000
11·0	·0047330	24·990	211·28	215·98	96,933	139,580,000	152·72	68,541	98,698,000
11·5	·0051608	27·249	193·77	225·80	101,340	145,930,000	159·67	71,657	103,190,000
12·0	·0056058	29·599	178·39	235·62	105,750	152,270,000	166·61	74,772	107,670,000
12·5	·0060780	32·091	164·53	245·44	110,150	158,620,000	173·55	77,888	112,160,000
13·0	·0065686	34·682	152·24	255·25	114,560	164,960,000	180·49	81,003	116,640,000
13·5	·0070778	37·371	141·29	265·07	118,960	171,300,000	187·43	84,118	121,130,000
14·0	·0076059	40·159	131·48	274·89	123,370	177,650,000	194·37	87,235	125,620,000
14·5	·0081525	43·045	122·66	284·71	127,780	184,000,000	201·32	90,350	130,100,000
15·0	·0087173	46·027	114·71	294·52	132,180	190,340,000	208·26	93,465	134,590,000

$d = 72$ inches $= 6$ feet

v	$s = \frac{h}{l}$	$s_m = 5{,}280\,\frac{h}{l}$	$G = \frac{l}{h}$	Q			Q'		
				Cubic Feet per Second	Gallons per Minute	Gallons per Day	Cubic Feet per Second	Gallons per Minute	Gallons per Day
.1	.00000035551	.0018771	2,812,900	2.8274	1,268.9	1,827,300	1.9993	897.26	1,292,100
.2	.0000014179	.0074865	705,260	5.6548	2,537.9	3,654,500	3.9985	1,794.5	2,584,100
.3	.0000031809	.016795	314,380	8.4822	3,806.8	5,481,800	5.9978	2,691.8	3,876,100
.4	.0000056385	.029771	177,350	11.310	5,075.7	7,309,000	7.9971	3,589.1	5,168,200
.5	.0000087842	.046380	113,840	14.137	6,344.7	9,136,300	9.9963	4,486.3	6,460,300
.6	.000012612	.066590	79,290	6.964	7,613.6	10,964,000	11.996	5,383.6	7,752,300
.7	.000017116	.090369	58,427	19.792	8,882.6	12,791,000	13.995	6,280.9	9,044,400
.8	.000022288	.11768	44,866	22.619	10,151	14,618,000	15.994	7,178.1	10,336,000
.9	.000028124	.14849	35,557	25.447	11,420	16,445,000	17.993	8,075.3	11,628,000
1.0	.000034619	.18278	28,886	28.274	12,689	18,273,000	19.993	8,972.6	12,921,000
1.1	.000041762	.22050	23,945	31.101	13,958	20,100,000	21.992	9,869.8	14,212,000
1.2	.000049552	.26163	20,181	33.929	15,227	21,927,000	23.991	10,767	15,505,000
1.3	.000058154	.30705	17,196	36.756	16,496	23,754,000	25.990	11,664	16,797,000
1.4	.000067243	.35504	14,871	39.584	17,765	25,582,000	27.990	12,562	18,089,000
1.5	.000077192	.40757	12,955	42.411	19,034	27,409,000	29.989	13,459	19,381,000

1.6	.000087561	.46232	11,421	45.239	20,303	29,236,000	31.988	14,356	20,673,000
1.7	.000098551	.52034	10,147	48.066	21,572	31,063,000	33.988	15,254	21,965,000
1.8	.00011048	.58335	9,051.1	50.893	22,841	32,890,000	35.987	16,151	23,257,000
1.9	.00012272	.64798	8,148.4	53.720	24,110	34,718,000	37.986	17,048	24,549,000
2.0	.00013598	.71799	7,353.8	56.548	25,379	36,545,000	39.985	17,945	25,841,000
2.1	.00014970	.79039	6,680.2	59.376	26,648	38,372,000	41.985	18,843	27,133,000
2.2	.00016416	.86678	6,091.4	62.203	27,916	40,199,000	43.984	19,740	28,425,000
2.3	.00017902	.94521	5,586.0	65.031	29,186	42,027,000	45.983	20,637	29,717,000
2.4	.00019447	1.0268	5,142.1	67.858	30,454	43;854,000	47.982	21,534	31,009,000
2.5	.00021053	1.1116	4,749.9	70.685	31,723	45,681,000	49.982	22,432	32,301,000
2.6	.00022719	1.1995	4,401.7	73.512	32,992	47,508,000	51.980	23,329	33,593,000
2.7	.00024443	1.2906	4,091.2	76.340	34,261	49,336,000	53.980	24,226	34,885,000
2.8	.00026207	1.3837	3,815.8	79.168	35,530	51,163,000	55.980	25,123	36,178,000
2.9	.00028047	1.4808	3,565.5	81.995	36,799	52,991,000	57.979	26,021	37,470,000
3.0	.00029944	1.5810	3,339.6	84.822	38,068	54,818,000	59.978	26,918	38,761,000
3.1	.00031948	1.6869	3,130.0	87.650	39,337	56,645,000	61.977	27,815	40,053,000
3.2	.00034017	1 7961	2,939.7	90.477	40,606	58,472,000	63.976	28,712	41,346,000
3.3	.00036175	1.9100	2,764 3	93.304	41,875	60,299,000	65.975	29,609	42,637,000
3.4	.00038372	2.0260	2,606.1	96.132	43,144	62,127,000	67.975	30,507	43,930,000
3.5	.00040630	2.1453	2,461.2	98.960	44,413	63,954,000	69.975	31,404	45,222,000

$d = 72$ *inches* $= 6$ *feet*

v	$s = \frac{h}{l}$	$s_m = 5{,}280\frac{h}{l}$	$G = \frac{l}{h}$	Q Cubic Feet per Second	Q Gallons per Minute	Q Gallons per Day	Q' Cubic Feet per Second	Q' Gallons per Minute	Q' Gallons per Day
3.6	.00042917	2.2660	2,330.1	101.79	45,681	65,781,000	71.973	32,301	46,514,000
3.7	.00045299	2.3918	2,207.5	104.61	46,950	67,608,000	73.972	33,199	47,806,000
3.8	.00047706	2.5188	2,096.2	107.44	48,219	69,435,000	75.971	34,096	49,098,000
3.9	.00050210	2.6511	1,991.6	110.27	49,488	71,262,000	77.970	34,993	50,389,000
4.0	.00052736	2.7845	1,896.2	113.10	50,757	73,090,000	79.971	35,891	51,682,000
4.1	.00055317	2.9207	1,807.8	115.92	52,026	74,917,000	81.969	36,787	52,974,000
4.2	.00057959	3.0602	1,725.4	118.75	53,295	76,745,000	83.969	37,685	54,266,000
4.3	.00060655	3.2026	1,648.7	121.58	54,564	78,572,000	85.969	38,582	55,558,000
4.4	.00063409	3.3480	1,577.1	124.41	55,833	80,399,000	87.967	39,479	56,850,000
4.5	.00066219	3.4963	1,510.1	127.23	57,102	82,226,000	89.966	40,377	58,142,000
4.6	.00069086	3.6477	1,447.5	130.06	58,371	84,054,000	91.967	41,274	59,435,000
4.7	.00072008	3.8020	1,388.7	132.89	59,640	85,882,000	93.966	42,172	60,727,000
4.8	.00074984	3.9591	1,333.6	135.72	60,909	87,708,000	95.964	43,069	62,018,000
4.9	.00078019	4.1194	1,281.7	138.54	62,178	89,536,000	97.965	43,966	63,311,000
5.0	.00081104	4.2822	1,233.0	141.37	63,447	91,363,000	99.963	44,863	64,603,000

5.5	.00097665	5.1567	1,023.9	155.51	69,791	100,500,000	109.96	49,349	71,062,000
6.0	.0011567	6.1073	864.53	169.64	76,136	109,640,000	119.96	53,836	77,523,000
6.5	.0013531	7.1445	739.03	183.78	82,480	118,770,000	129.95	58,322	83,983,000
7.0	.0015643	8.2592	639.28	197.92	88,826	127,910,000	139.95	62,809	90,444,000
7.5	.0017840	9.4195	560.53	212.06	95,170	137,040,000	149.94	67,295	96,904,000
8.0	.0020166	10.647	495.90	226.19	101,510	146,180,000	159.94	71,781	103,360,000
8.5	.0022690	11.980	440.72	240.33	107,860	155,320,000	169.94	76,268	109,820,000
9.0	.0025354	13.387	394.41	254.47	114,200	164,450,000	179.93	80,753	116,280,000
9.5	.0028156	14.866	355.16	268.60	120,550	173,590,000	189.93	85,239	122,740,000
10.0	.0031094	16.418	321.60	282.74	126,890	182,730,000	199.93	89,726	129,210,000
10.5	.0034253	18.085	291.94	296.88	133,240	191,860,000	209.92	94,213	135,670,000
11.0	.0037561	19.832	266.23	311.01	139,580	201,000,000	219.92	98,698	142,120,000
11.5	.0041019	21.658	243.78	325.15	145,930	210,140,000	229.92	103,190	148,590,000
12.0	.0044626	23.562	224.08	339.29	152,270	219,270,000	239.91	107,670	155,050,000
12.5	.0048383	25.546	206.69	353.43	158,620	228,410,000	249.91	112,160	161,510,000
13.0	.0052285	27.606	191.26	367.56	164,960	237,540,000	259.90	116,640	167,970,000
13.5	.0056338	29.746	177.50	381.70	171,300	246,680,000	269.90	121,130	174,430,000
14.0	.0060540	31.965	165.18	395.84	177,650	255,820,000	279.90	125,620	180,890,000
14.5	.0064886	34.259	154.12	409.98	184,000	264,950,000	289.89	130,100	187,350,000
15.0	.0069379	36.632	144.14	424.11	190,340	274,090,000	299.89	134,590	193,810,000

INDEX

NOTE.—All items in this index refer first to the section (see the Preface) and then to the page of the section. Thus, "Aeration, §91, p33," means that aeration will be found on page 33 of section 91.

A

Absorption-ditch system of broad irrigation, §92, p38.
of water in canals and reservoirs, §93, p18.
Absorptive power of the soil in intermittent filtration, §92, p40.
Accuracy of sewerage calculations, §88, p16.
Acre-foot, §93, p5.
Aeration, §91, p33.
Aerobic bacteria, §92, p10.
Air inlets for sewers, §89, p9.
lift, §84, p41.
-locked pipe, §85, p31.
valves, §85, p31.
Albuminoid ammonia in sewage, §92, p9.
ammonia in water, §90, p19.
Alfalfa raising by irrigation, §93, p63.
Algæ in water, §90, p24.
Alkali in irrigation waters, §93, p8.
Remedies for, §93, pp8, 9.
Ammonia, Albuminoid, in sewage, §92, p9.
Albuminoid, in water, §90, p19.
Free, in sewage, §92, p9.
Free, in water, §90, p20.
Anaerobic bacteria, §92, p10.
Analyses of water, Interpretation of, §90, pp14, 22.
Sewage, Interpretation of, §92, p5.
Analysis, Chemical, of sewage, §92, p5.
of water, Bacteriological, §90, p23.
Sewage, §92, p1.
Water, Chemical, §90, pp17, 22.
Water, Collecting samples for, §90, p15.
Anchorage for elevated tanks, §86, p57.
for standpipes, §86, p36.
Aneroid barometer, Use of, in reservoir reconnaissance, §86, p3.
Appropriation, Priority of, §93, p65.
Artesian wells, §84, p33.
Artificial irrigation, §93, p2.
Ash in sewage after combustion, §92 p8

B

Back filling of sewer trenches, §89, p38.
filling of sewer trenches, Cost of, §89, p54.
Bacteria, §90, p8; §92, p10.
Action of, in broad irrigation, §92, p30.
Action of, in the purification of sewage, §92, pp24, 26.
Aerobic, §92, p10.
Anaerobic, §92, p10.
beds, §92, p50.
Distribution of, at different depths, §92, p30.
Effect of dilution of sewage on, §92, p18.
Facultative, §92, p10.
Measurement of, §92, p12.
Non-pathogenic, §92, pp11, 12.
Parasitic, §92, p11.
Pathogenic, §92, pp11, 12.
Presence of, in water, air, etc., §92, p11.
Proportion of, removed by filtration, §91, p31.
Saprophytic, §92, p11.
Saprophytic, Beneficial effect of, §92, p12.
Bacterial analysis of sewage, §92, p12.
pollution of water, §90, p8.
Bacteriological analysis of water, §90, p23.
Barometer, Aneroid, Use of, in reservoir reconnaissance, §86, p3.
Barometric leveling for storage reservoirs, §86, p3.
Barrel, Working, of a deep-well pump, §84, p43.
Basins, Catch, §89, p11.
Settling, §90, p25.
Battery of wells, §84, p37.
Bed-and-ridge system of broad irrigation, §92, p37.
Sand, for filters, §91, p2.
Beds, Bacteria, §92, p50.
Contact, §92, pp52, 64.
Contact, Multiple-, §92, p56.

136—41

Beds—(Continued)
Filter, Cleaning of, §91, p21.
Filter, for intermittent filtration, §92, p40.
for slow sand filters, §91, p7.
Bell of water-supply cast-iron pipe, §85, p17.
Bench marks in sewerage work, §87, p5.
Bends for water-supply pipes, §85, p30.
Beneficial use, Principle of, §93, p65.
Biological nature of filtration, §91, p3.
purification of sewage, §92, p24.
Blocks and wedges under water-supply pipe, §85, p19.
Invert, for brick sewers, §88, p39.
Blow-offs in water-supply pipes, §85, p42.
Boston, Evaporation in, Table of, §84, p72.
Table of water consumption in, §84, p65.
Bottom plates of standpipes, §86, p35.
Bracing, Wind, for elevated tanks, §86, p57.
Branches, House, Size of, §89, p26.
Pipe, Problems on, §84, p54.
Sewer Y, for house connections, §89, p24.
Brick for sewers, Quality of, §88, p37.
-sewer intersections, §89, p26.
sewers, §88, p36.
sewers, Circular, Table of velocities and discharges for, §88, p45.
sewers, Construction of, §89, p16.
sewers, Cost of, §89, p50.
sewers, Cross-section of, §88, p36.
sewers, Estimates for, §89, p50.
sewers, Foundations for, §89, p13.
sewers, Thickness of, §88, p37.
Bricks, Number of, required for brick sewers, §89, p50.
Number of, required for manholes, §89, p54.
Bridge crossings for water-supply pipe, §85, p23.
Bridges for flumes, §93, p50.
for sewers, §89, p22.
Broad irrigation, §92, p28.
irrigation, Advantages and disadvantages of, §92, p62.
irrigation, Crops suited to, §92, p33.
irrigation, Effect of, on health, §92, p32.
irrigation, Effect of temperature on, §92, p34.
irrigation, Objections to, §92, p29.
irrigation, Systems of, §92, p35.
Buerkli's formula for effluent, §87, p42.
Butt joints, §86, p31.

C

Calculations, Sewer, §88, p9.
Sewer, Accuracy of, §88, p16.
Canals, Culverts for, §93, p42.
Drops in, §93, p42.
Earthen, §93, p31.
Canals—(Continued)
Earthen, Cross-section of, §93, p32.
Earthen, Effect of depth on slope of, §93, p33.
Earthen, Formula for flow in, §93, p31.
Earthen, General remarks on, §93, p34.
Earthen, Limiting velocity in, §93, p32.
Emptying sluices for, §93, p41.
Falls in, §93, p42.
for irrigation, §93, p28.
for irrigation, Grades for, §93, p30.
for irrigation, Location of, §93, p29.
for irrigation, Surveys for, §93, p28.
Lined, §93, p35.
lined with concrete, §93, p37.
lined with dry stone, §93, p35.
lined with rubble masonry, §93, p37.
Overflows for, §93, p40.
Turnouts for, §93, p44.
Capacity of contact beds, §92, p53.
of distributing reservoirs, §86, p16.
of septic tanks, §92, p47.
of standpipes, §86, p20.
Relative, of egg-shaped and circular sewers, §88, p24.
Carbonate, Ferrous, Removal of, from water, §91, p33.
Cast-iron pipe for water supply, §85, p43.
-iron pipe for water supply, Thickness of, §85, p43.
-iron pipe for water supply, Weight of, §85, p44.
-iron pipe line, Weight of, §85, p46.
iron, Use of, in sewer construction, §88, p41.
-iron water-supply pipe, Laying of, §85, p17.
Castings, Special, for water-supply pipe lines, §85, p29.
Catch basins, §89, p11.
basins, Cost of, §89, p55.
-work system of broad irrigation, §92, p37.
Cement, Amount of, for laying sewer pipe, §89, p49.
joints for sewer pipes, §88, p34.
sewers, §88, p40.
Chalybeate waters, §90, p10.
Change in section of sewer, Loss of grade from, §87, p13.
Check-system of irrigation, §93, p57.
-valves, §85, p34.
Checker-board system of irrigation, §93, p57.
Chemical analysis of sewage, §92, p5.
precipitation, Disadvantages of, §92, p62.
precipitation in sewage purification, §92, p19.
-precipitation plant at Worcester, §92, p23.

Chemical—(Continued)
purification of manufacturing wastes, §92, p19.
purification of water, §91, p31.
water analysis, §90, pp17, 22.
Chlorine in sewage, §92, p9.
in water, §90, p18.
Circular and egg-shaped sewers, Relative capacity of, §88, p24.
horizontal girders for elevated tanks, §86, p52.
sewers, Calculations for, §88, p9.
vertical girder for elevated tank, §86, p50.
Clark degree of hardness, §90, p11.
Classification of reservoirs, §86, p1.
Cleaning of filter beds, §91, p21.
of settling basins, §90, p29.
Coagulants, §90, p30.
Coagulating apparatus for mechanical filters, §91, p29.
Coagulation, §90, p30.
Coarse filters, Distribution of sewage on, §92, p52.
filters for sewage filtration, §92, p50.
Coating of water-supply pipes, §84, p63.
Coefficient, Discharge, for gravel, §91, p11.
of storm flow, §87, p29.
Uniformity, of sand, §91, p9.
Coke, Straining of sewage through, §92, p57.
Color in water, Permissible, §90, p6.
of water, §90, p6.
of water, Measurement of, §90, p7.
Columns for elevated tanks, §86, pp49, 55.
Combined sewerage system, §87, p2.
system of sewers, Outlets for, §89, p32.
Combustible matter in sewage, §92, p8.
Comparison of sewerage systems, §87, p2.
Compensation, Stream, §86, p13.
Composition of sewage, §92, p1.
Compound pipe line, §84, p59.
Concrete, Canals lined with, §93, p37.
flumes for irrigation, §93, p49.
for sewers, Quality of, §88, p40.
sewers, Construction of, §89, p17.
sewers, Thickness of, §88, p39.
Conduits, Irrigation, §93 p28.
Irrigation, Tunnels for, §93, p53.
Connection between sewers on different grades, §89, p7.
Connections, House, in water-supply systems, §85, p31.
House, to sewers, §89, p4.
Construction of brick sewers, §89, p16.
of concrete sewers, §89, p17.
of shallow wells, §84, p34.
of standpipes, §86, p27.
of water-supply pipe lines, §85, p13.
Construction—(Continued)
Sewer, §89, p1.
Consumption, Water (See *Water*).
Contact beds, §92, pp52, 64.
beds, Capacity of, §92, p52.
beds, Multiple-, §92, p66.
Contemporary flow, Formula for, §87, p31.
flow of storm water, §87, p27.
Continental gravity filter, §91, p24.
Continuous-flow process for softening hard water, §91, p36.
-flow settling basins, §90, p26.
-flow settling basins, Velocity in, §90, p28.
Copper sulphate, Effect of, on fish, §91, p31.
-sulphate treatment of water, §91, p31.
Corn raising by irrigation, §93, p64.
Cost of back filling of sewer trenches, §89, p54.
of brick sewers, §89, p50.
of catch basins, §89, p55.
of earthwork in sewer construction, §89, p52.
of filters, §91, p29.
of filtration, §91, p29.
of flush tanks, §89, p55.
of irrigation works, §93, p3.
of laying sewer pipe, §89, p49.
of lumber, §89, p55.
of manholes, §89, p54.
of operating pumping machinery for water supply, §85, p59.
of pipe sewers, §89, p48.
of pumping from sewer trenches, §89, p53.
of pumping machinery for water supply, §85, pp57, 59.
of pumping water, §85, p61.
of rock excavation for sewers, §89, p53.
of sheeting for sewer trenches, §89, p53.
of sundry sewer accessories, §89, p55.
of windmills, §93, p21.
Costs of various types of engines, Table of, §85, p58.
Covering of slow sand filters, §91, p18.
Covers for lamp holes, §89, p10.
Manhole, §89, p5.
Cradles for brick sewers, §89, p13.
Crenothrix, §90, p11.
Crops, Amount of water required for, §93, p2.
Depth of irrigation required for, §93, p6.
Effect of irrigation on, §93, p3.
Irrigating periods for, §93, p7.
Necessity of water for, §93, p1.
Raising of, §93, p62.
suited for sewage irrigation, §92, p33.
Cross-connecting pipe, §85, p30.
Crossings, Sewer, §89, p19.
Water-supply pipe at, §85, pp20, 23.
Cross-section of brick sewers, §88, p36.

Cross—(Continued)
-section of earthen canals, §93, p32.
-sections for sewers, §88, p20.
Culverts for irrigation canals, §93, p42.
Current-meter measurements for irrigation, §93, p16.
Curves in sewer lines, §89, p42.
in water-supply pipes, §85, p24.
Cutting, Pipe, §85, p26.

D

Dam for storage reservoir, Height of, §86, p15.
for storage reservoir, Surveying the site for, §86, p12.
Dams for irrigation work, §93, p23.
Loose-rock, §93, p27.
Reinforced-concrete, §93, p26.
Timber, §93, p23.
Data relating to rainfall, §87, p16.
Datum plane in sewerage work, §87, p5.
plane in water-supply work, §84, p53.
Dead ends, §85, p6.
Deep-and-wide socket for sewer pipe, §88, p31.
well, Estimating probable yield of, §84, p36.
-well pumps, §84, p43.
wells, §84, pp33, 36.
wells, Advantages and disadvantages of, §84, p45.
wells, Methods of delivery from, §84, p41.
wells, Operation of, §84, p46.
Degree, Clark, of hardness, §90, p11.
Frankland, of hardness, §90, p11.
Depth of flow in sewers, Effect of, on velocity, §88, p17.
of sewers, §87, p14.
of water-supply pipe, §85, p16.
Design of sewers, General remarks on, §88, p19.
of standpipes, §86, p27.
of timber flumes, §93, p47.
of water-supply pipe systems, §85, pp5, 9.
Detritus in sewage, §92, p3.
Diagrams for sewer calculations, §88, p13.
Diameters of sewer pipes, §88, p30.
of standpipes, §86, p20.
Diarrheal diseases, §92, p13.
Dilution of sewage, Effect of, on bacteria, §92, p18.
Sewage purification by, §92, pp17, 62.
Dimensions of egg-shaped sewers, §88, p27.
Direct-pressure system of water distribution §85, p3.
Discharge coefficients for gravel, §91, p11.
Diseases, Diarrheal, §92, p13.
Dissemination of, by water, §92, p14.
Infectious, §92, p11.
Infectious, Dissemination of, §92, p13.

Diseases—(Continued)
Zymotic, §92, p11.
Distributing-reservoir system, §85, p2.
reservoirs, §86, pp1, 15.
reservoirs, Advantages of, §86, p15.
reservoirs, Capacity of, §86, p16.
reservoirs, Location of, §86, p15.
reservoirs, Pipe connections in, §86, p17.
Distribution of water, Methods of, §85, p1.
Diverter, Mechanical, §89, p30.
Diverting weir, §89, p28.
Double-strength sewer pipe, §88, p31.
Drainage district, Area of, §87, p39.
district, Form of, §87, p38.
to prevent alkalinity, §93, p8.
Drilling machines for wells, §84, p37.
Driving of deep wells, §84, p37.
Drops in canals, §93, p42.
Dug wells, §84, p33.
Duty of pumping engines, §85, p60.
of sewage, §92, p31.
of water used for irrigation, §93, p5.

E

Earthen canals, Cross-section of, §93, p32.
canals, Effect of depth on slope of, §93, p33.
canals, Formula for flow in, §93, p31.
canals, General remarks on, §93, p34.
canals, Limiting velocity in, §93, p32.
Earthwork in sewer construction Cost of, §89, p52.
Efficiency of pumping engines, §85, p60.
of pumps, §85, p60.
Effluent, §87, p39.
Formulas for total, §87, pp39, 42.
Egg-shaped sewers, §88, p20.
-shaped sewers, Capacity of, compared with circular sewers, §88, p24.
-shaped sewers, Elements of cross-section of, §88, p23.
-shaped sewers, Formulas for, §88, p20.
-shaped sewers, General form of, §88, p20.
-shaped sewers, New form of, §88, p23.
-shaped sewers, Old form of, §88, p23.
-shaped sewers, Practical dimensions of, §88, p27.
-shaped sewers, Table of velocities and discharges for, §88, p46.
Elbows for water-supply pipes, §85, p30.
Elevated tanks, §86, p45.
tanks, Anchorage for, §86, p57.
tanks, Conditions proper for, §86, p45.
tanks, Forms of, §86, p46.
tanks, Foundations for, §86, p56.
tanks, Roofs for, §86, p59.
tanks, Supports for, §86, p48.
tanks, Thickness of bottom of, §86, p36.

Elevated—(Continued)
tanks, Wind bracing for, §86, p57.
tanks, Wind pressure on, §86, p57.
Ends, Dead, §85, p6.
Engines, Costs of various types of, §85, p58.
Escapes for irrigation canals, §93, p41.
Estimates for brick sewers, §89, p50.
for pipe sewers, §89, p48.
Evaporation as affecting water supply, §84, p23.
in Boston and Rochester, Table of, §84, p72.
Measurement of, §93, p16.
Quantity of, §93, p16.
Excavation for sewers, §89, pp33, 37.
for water-supply pipe, §85, p15.
Rock, for sewers, Cost of, §89, p53.
Excrement, Human, Composition of, §92, p2.

F

Facultative bacteria, §92, p10.
Falls in canals, §93, p42.
Fan plan for sewerage systems, §88, p3.
Feces, Amount of, in human excrement, §92, p2.
Fermentation of sewage, §92, p26.
Ferrous carbonate, Removal of, from water, §91, p33.
sulphate, Removal of, from water, §91, p33.
Fertilizing value of sewage, §92, p32.
Fill-and-draw settling basins, §90, pp26, 27.
Filling, Back, of sewer trenches, §89, p38.
water-supply pipes, §85, p28.
Filter beds, Cleaning of, §91, p21.
beds for intermittent filtration, §92, p40.
beds for slow sand filters, §91, p7.
Continental gravity, §91, p24.
Jewell gravity, §91, p26.
Jewell pressure, §91, p27.
sand, Washing of, §91, p22.
tanks, Material for, §91, p27.
Filters, §91, p1.
Capacity of, §91, p3.
Coarse, Distribution of sewage on, §92, p52.
Coarse, for sewage filtration, §92, p50.
Cost of, §91, p29.
Gravity, §91, pp23, 24.
Kinds of, §91, p1.
Laying sand courses for, §91, p14.
Loss of head in, §91, pp10, 12.
Mechanical, Classification of, §91, p23.
Mechanical, Coagulating apparatus for, §91, p29.
Mechanical, General description of, §91, p2.
Pressure, §91, pp23, 27.
Sand bed for, §91, p2.
Sand for, §91, p8.
Filters—(Continued)
Slow sand, Construction of, §91, p4.
Slow sand, Covering of, §91, p18.
Slow sand, General description of, §91, p1.
Slow sand, Operation of, §91, p21.
Slow sand, Outflow from, §91, p15.
Sprinkling, §92, p58.
Filtration, §91, p1.
Biological nature of, §91, p3.
Combined methods of, §91, p30.
Cost of, §91, p29.
Impurities removed by different methods of, §91, p30.
Intermittent, Advantages and disadvantages of, §92, p63.
of sewage, §92, pp24, 25.
of sewage, Intermittent, §92, pp25, 39.
of sewage, Intermittent, Principle of, §92, p37.
of sewage through coarse filters, §92, p50.
plants, Pure-water reservoirs for, §91, p5.
Rate of, §91, p2.
Upward, Advantages and disadvantages of, §2, p63.
Upward, Tanks, §92, p49.
Fire-hose, §85, p39.
-streams, Depletion of standpipes by, §86, p22.
Fires, Water required for, §84, pp9, 67.
Fish, Effect of copper sulphate on, §91, p31.
test of sewage, §92, p16.
Flange steel, §86, p28.
Flanged pipe, §85, p47.
Flexible joints, §85, p20.
Flooding, Irrigation by, §93, p56.
Flow at inlet of sewer, §87, p33.
at points below inlet of sewer, §87, p33.
Contemporary, Formula for, §87, p31.
from a watershed, Average, §84, p30.
from a watershed, Minimum, §84, p30.
in earthen canals, Formula for, §93, p31.
in riveted pipe, §85, p49.
in sewers, Maximum, §87, p24.
in sewers, Maximum rate of, §87, p28.
in storm-water sewers, Fluctuations of, §88, p20.
in timber flumes, §93, p47.
into sewer per acre, Practical formula for, §87, p36.
into sewers at different points, Formulas for, §87, p43.
into sewers, Formula for rate of, §87, p31.
into sewers, Talbot's formula for rate of, §87, p34.
line of a reservoir, §86, pp11, 12.
of storm water on different surfaces, §87, p25.

Flow—(Continued)
of storm water per acre, Formula for, §87, p32.
of streams, §84, p25.
of streams, Gauging of, for irrigation, §93, p15.
of water in lined canals, §93, p36.
of water, Resistance of gravel to, §91, p11.
of water, Resistance of sand to, §91, p10.
Storm, Coefficient of, §87, p29.
through cast-iron pipes, Table of approximate formulas for, §84, p74.
Flumes, Bridges for, §93, p50.
Concrete, for irrigation, §93, p49.
for irrigation, §93, p45.
Iron, for irrigation, §93, p48.
Timber, §93, p45.
Timber, Design of, §93, p47.
Timber, Flow in, §93, p47.
Trestles for, §93, p50.
Flush tanks, §89, p38.
tanks, Cost of, §89, p55.
Foote's water meter, §93, p60.
Foundations for elevated tanks, §86, p56.
for sewers, §89, p12.
for standpipes, §86, p38.
Timber, for sewers, §89, p15.
Frames for lamp holes, §89, p10.
Manhole, §89, p5.
Frankland degree of hardness, §90, p11.
Free ammonia in sewage, §92, p9.
ammonia in water, §90, p20.
Fund, Reclamation, §93, p66.
Furrows, Method of irrigation by, §93, p58.

G

Gauge, Rain, §84, p16.
Gauging of streams, §84, p25.
of streams for irrigation, §93, p15.
Gaugings of London sewers, §87, p28.
of sewage, §87, p49.
Girder, Vertical circular, for elevated tank, §86, p50.
Girders, Horizontal circular, for elevated tanks, §86, p52.
Grade in sewers, Loss of, from change in size, §87, p13.
of house sewer branches, §89, p36.
stakes for sewers, §89, p44.
Grades for irrigation canals, §93, p30.
for pipe sewers, Minimum, §87, p12.
for sewers, §87, p11.
Grading for sewers, §89, p44
Grasses, Raising of, by irrigation, §93, p64.
Gravel, Resistance of, to flow of water, §91, p11.
Table of discharge coefficients for, §91, p11.
Gravity filters, §91, pp23, 24.
system of water distribution, §85, p1.
Gridiron system of water distribution, §85, p6.
Ground water, Sources of, §93, p19.

H

Hammer test of water-supply pipe, §85, p14.
Hard water, §90, p11.
water, Objections to, §90, p13.
water, Permanently, Softening of, §91, p35.
water, Softening of, §91, p34.
water, Temporarily, Softening of, §91, p34.
Hardness, Clark degree of, §90, p11.
Frankland degree of, §90, p11.
of water, Causes of, §90, p11.
of water, Measurement of, §90, p11.
of water, Permanent, §90, p11.
of water, Temporary, §90, p11.
of water, Permissible, §90, p14.
of water, Soap test for, §90, p12.
Head, Loss of, in filters, §91, pp10, 12.
-works for canals, §93, p38.
Health, Effect of sewage irrigation on, §92, p32.
Height of standpipes, §86, p20.
Holes, Lamp, §89, p9.
Hose, Fire-, §85, p39.
House connections in water-supply systems, §85, p31.
connections, Sewer, §89, p24.
sewer branches, Grade of, §89, p26.
sewer branches, Size of, §89, p26.
sewers, §87, p2.
Household waste in sewage, §92, p3.
Hub of water-supply cast-iron pipe, §85, p17.
Hydrants, §85, p38.
Table of pressures required for, §85, p40.
Hydraulic radii of egg-shaped and circular sewers compared, §88, p25.

I

Ice as a vehicle of infection, §92, p14.
Effect of, on standpipes, §86, p44.
Ignition, Loss on, §92, p8.
Impervious area, Relative, §87, p26.
Impurities in water, §90, p1.
in water compared with those in sewage, §92, p6.
removed from water by different filtration processes, §91, p30.
Increaser, §85, p29.
Incubation test of sewage, §92, p16.
Infection and dissemination of diseases, §92, p13.
and pollution, §92, p15.
Conveyance of, by ice, §92, p14.
Conveyance of, by water, §92, p14.

Infection—(Continued)
Sources of, §92, p11.
Infectious diseases, §92, p11.
Inlet of sewer, Flow at, §87, p33.
Inlets, Air, for sewers, §89, p9.
Raw-water, for slow sand filters, §91, p14.
Inorganic matter, §92, p1.
matter in sewage, §92, p1.
Inspection of pipe for water supply, §85, p14.
Intakes, §84, p50.
Intercepting plan for sewerage systems, §88, p3.
Intermittent-filter plant, §92, p43.
filtration, Advantages and disadvantages of, §92, p63.
filtration, Beds for, §92, p40.
filtration in winter, §92, p43.
filtration of sewage, §92, pp25, 39.
filtration of sewage, Principle of, §92, p39.
filtration, Rate of, §92, p41.
process for softening hard water, §91, p36.
Intersections, Brick-sewer, §89, p26.
Invert blocks for brick sewers, §88, p39.
Inverted siphons, §89, p19.
Iron flumes for irrigation, §93, p48.
-impregnated waters, §90, p10.
Removal of, from ground water, §91, p33.
Use of, in sewer construction, §88, p41.
Irrigating periods, §93, p7.
Irrigation, Artificial, §93, p2.
as affected by rainfall, §93, p13.
Broad, §92, p28.
Broad, Advantages and disadvantages of, §92, p62.
Broad, Amount of sewage required for, §92, p32.
Broad, Crops suited to, §92, p33.
Broad, Effect of, on health, §92, p32.
Broad, Effect of mperature on, §92, p34.
Broad, Objections to, §92, p29.
by flooding, §93, p56.
by furrows, §93, p58.
by sprinkling, §93, p54.
Canals for, §93, p28.
Check-system of, §93, p57.
Checker-board system of, §93, p57.
Conduits for, §93, p28.
conduits, Tunnels for, §93, p53.
Depth of, necessary for various crops, §93, p6.
Duty of water used for, §93, p5.
Effect of, on crops, §93, p3.
Effect of, on the value of land, §93, p3.
Extent of, in different countries, §93, p1.
Flumes for, §93, p45.
General methods of, §93, p54.
Ground water for, §93, p19.
Irrigation—(Continued)
laws, §93, p65.
Natural, §93, p2.
only one factor in cultivation, §93, p4.
Preliminary surveys for, §93, p12.
Preparation of the soil for, §93, p54.
Quantity of water for, §93, p11.
Reservoirs for, §93, p21.
Sources of water for, §93, p11.
Subsoil, §93, p58.
Subsurface, §92, p59.
Surface water for, §93, p11.
Units used in measuring water for, §93, p5.
Water storage for, §93, p21.
water, Weir measurements of, §93, p60.
works, Cost of, §93, p3.
works, Dams for, §93, p23.

J

Jewell gravity filter, §91, p26.
pressure filter, §91, p27.
Joint, Stanford, for sewer pipes, §88, p35.
Joints between standpipe plates, §86, p29.
Butt, §86, p31.
Cement, for sewer pipes, §88, p34.
Flexible, §85, p20.
for cast-iron water-supply pipe, §85, p17.
for riveted water-supply pipe, §85, p48.
for sewer pipe, §88, pp31, 34.
for standpipes, §86, p34.
Horizontal, for standpipes, §86, p33.
Lap, §86, p29.
Lap, Double-riveted, §86, p30.
Lap, Single-riveted, §86, p30.
Lap, Triple-riveted, §86, p31.
Lead, Removing, §85, p25.
Vertical, for standpipes, §86, p33.

K

Kutter's formula, Application of, to sewers, §88, p10.

L

Lake intakes, §84, p50.
Lakes, Pollution of, §90, p1.
Lamp holes, §89, p9.
holes, Covers and frames for, §89, p10.
Lap joints, §86, p29.
joints, Double-riveted, §86, p30.
joints, Single-riveted, §86, p30.
joints, Triple-riveted, §86, p31.
-welded pipe, §85, p50.
Laying cast-iron water-supply pipe at crossings, §85, pp20, 23.
of cast-iron water-supply pipe, §85, p17.
of sewer pipe, Cost of, §89, p49.
of riveted steel pipe, §85, p49.

Laying—(Continued)
of wooden-stave pipe, §85, p55.
sand courses for filters, §91, p13.
Law, Reclamation, §93, p66.
Laws on irrigation, §93, p65.
Lead, Weight of, in pipe joints, §85, p20.
Leaks in water-supply pipes, §85, p26.
Leaping weir, §89, p29.
Length, Pipe, §85, p17.
Lengths of sewer pipes, §88, p30.
Leveling, Barometric, for storage reservoirs, §86, p3.
for sewer work, §89, p43.
for sewers, §87, p5.
Lift, Air, §84, p41.
Limiting velocity in earthen canals, §93, p32.
Line, Flow, of a reservoir, §86, pp11, 12.
Pipe, Compound, §84, p59.
Lined canals, §93, p35.
Location of distributing reservoirs, §86, p15.
of irrigation canals, §93, p28.
of manholes, §89, p1.
of settling basins, §90, p29.
of sewers, §87, pp8, 11.
of standpipes, §86, p19.
of storage reservoirs, §86, p2.
of storage reservoirs for irrigation, §93, p22.
of water-supply pipes, §85, p6.
London sewers, Gaugings of, §87, p28.
Long pipes, Flow of water in, §84, p52.
Loose-rock dams, §93, p27.
Loss of head in filters, §91, pp10, 12.
Losses of water used for irrigation, §93, pp16, 18.
Lumber, Cost of, §89, p55.

M

Machines, Drilling, for wells, §84, p37.
Manhole covers, §89, p5.
frames, §89, p5.
walls, Thickness of, §89, p4.
Manholes, §89, p1.
Cost of, §89, p54.
for connecting sewers on different grades, §89, p7.
for large sewers, §89, p8.
Location of, §89, p1.
Number of bricks required for, §89, p54.
Shape of, §89, p3.
Sizes of, §89, p5.
Manufacturing, Quantity of water used for, §84, p5.
wastes, Chemical purification of, §92, p19
wastes in sewage, §92, p3.
Map, Working, for sewer construction, §89, p44.
Maps for sewer location, §87, p8.
for water supply, §85, p5.
Masonry, Rubble, Canals lined with, §93, p37.
Massachusetts experimental station for sewage purification, §92, p24.
Material for filter tanks, §91, p27.
for water-supply steel pipe, §85, p48.
Materials for settling basins, §90, p27.
for sewage coarse filters, §92, p50.
for standpipes, §86, p27.
used for sewers, §88, pp28, 36.
Matter, Inorganic, §92, p1.
Organic, §92, p1.
Maximum rate of flow in sewers, §87, p28.
McMath's formula for effluent, §87, p44.
Measurement of water furnished for irrigation, §93, p59.
Mechanical diverter, §89, p30.
filters, Classification of, §91, p23.
filters, General description of, §91, p2.
Metallic substances in water, §90, p17.
Meter, Venturi, §85, p35.
Water, Foote's, §93, p60.
Meters, Table of water consumption as affected by, §84, p66.
Water, §85, p33.
Water, Service, §85, p36.
Microscopic examination of water, §90, p23.
Minimum grades for pipe sewers, §87, p12.
Modules, §93, p60.
Mortar, Ingredients of, §89, p51.
necessary for brick sewers, §89, p50.

N

Natural irrigation, §93, p2.
Nitrates in sewage, §92, p10.
in water, §90, p21.
Nitrites in sewage, §92, p10.
in water, §90, p21.
Nitrogen, Organic, in sewage, §92, p9.
Non-pathogenic bacteria, §92, p11.
Notes, Sewer-construction, §89, p46.
Transit, of sewer surveys, §89, p42.
Nozzles for fire-hose, §85, p39.

O

Oats, Raising of, by irrigation, §93, p64.
Odors in water, §90, p9.
in water, Removal of, by filtration, §91, p31.
Operation of slow sand filters, §91, p21.
Organic matter, §92, p1.
matter in sewage, §92, p1.
matter in sewage, Sources of, §92, p7.
matter in water, §90, p19.
matter, Reduction of, in sewage purification, §92, p10.

Organic—(Continued)
nitrogen in sewage, §92, p9.
Outflow from slow sand filters, §91, p15.
Outlet pipes for storage reservoirs, §86, p14.
Outlets for sewers, §89, p31.
Overflows for irrigation canals, §93, p40.
for sewers, §89, p28.
Oxidation of sewage by aeration, §92, p27.
Purification of sewage by, §92, p17.
Oxygen consumed by, §92, p9.
Oxygen-consuming capacity of water, §90, p22.

P

Painting of standpipes, §86, p46.
Parasitic bacteria, §92, p11.
Pathogenic bacteria, §92, p11.
Percolating-filter treatment of sewage, §92, p58.
Percolation as affecting water supply, §84 p23.
Periods, Irrigating, §93, p7.
Service, in irrigation, §93, p7.
Perpendicular plan for sewerage system, §88, p1.
Physical tests of water, §90, p14.
Piling, Sheet, for sewer trenches, §89, p34.
Pipe-and-hydrant system of broad irrigation, §92, p36.
branches, Problems on, §84, p54.
Cast-iron, for water supply, §85, p43.
coating, §84, p63.
connections in distributing reservoirs, §86, p15.
cutting, §85, p26.
Flanged, §85, p47.
for water supply, Distributing and handling of, §85, p13.
for water supply, Inspection of, §85 p14.
Lap-welded §85, p50.
length, §85, p17.
line, Cast-iron, Weight of, §85, p46.
line, Compound, §84, p59.
lines for water supply, Construction of, §85, p13.
sections, §85, p17.
Sewer, Double-strength, §88, p31.
Sewer, Joints for, §88, pp31, 34.
Sewer, Requisites of, §88, p32.
Sewer, Standard, §88, p31.
Sewer, Strength of, §88, p31.
Sewer, Table of thickness of, §88, p47.
Sewer, Thickness of, §88, p31.
sewers, Cost of, §89, p48.
sewers, Estimates for, §89, p48.
sewers, Foundations for, §89, p12.
sewers, Table of velocities and discharges for, §88, p42.
Pipe—(Continued)
Spiral riveted, §85, p50.
Steel, for water supply, §85, p47.
system for water supply, Design of, §85, pp5, 9.
Terra-cotta, General description of, §88, p28.
Terra-cotta, Making of, §88, p29.
Water-supply, Cast-iron, Laying of, §85, p17.
Water-supply, Depth of, §85, p16.
Water-supply, Excavation for, §85, p15.
Water-supply, Kinds and properties of, §85, p43.
Wooden-stave, §85, p51.
Pipes, Long, Flow of water in, §84, p52.
Outlet, for storage reservoirs, §86, p14.
Service, §85, p55.
Sewer, Standard dimensions of, §88, p30.
Sewer, Standard forms of, §88, p34.
Table of approximate formulas for flow in, §84, p74.
Use of, in irrigation work, §93, p53.
Water-supply, Location and size of, §85, p6.
Water-supply, Pressure in, §85, p3.
Water-supply, Roughness of, §84, p62.
Water-supply, Submerged, §85, p20.
Plan, Fan, for sewerage systems, §88, p3.
Intercepting, for sewerage systems, §88, p3.
Perpendicular, for sewerage systems, §88, p1.
Radial, for sewerage systems, §88, p6.
Zone, for sewerage systems, §88, p6.
Plane, Datum, in sewerage work, §87, p5.
Plaster plates, §93, p51.
Plates, Bottom, of standpipes, §86, p35.
Plaster, §93, p51.
Platinum-cobalt method of measuring color in water, §90, p7.
Pollution and infection, §92, p15.
of lakes, §90, p1.
of rivers, §90, p1.
of water, Bacterial, §90, p8.
Population, Probable future, §84, p8.
Water consumption according to, Table of, §84, p65.
Power for pumping, Best, §93, p20.
required in pumping through water-supply mains, §85, p55.
Precipitants for sewage, §92, p19.
Precipitation, Chemical, Disadvantages of, §92, p62.
plants for sewage treatment, §92, p20.
Pressure, Direct, system of water distribution, §85, p3.
filters, §91, pp23, 27.
in water-supply pipes, §85, p3.

Pressure—(Continued)
Wind, on elevated tanks, §86, p40.
Wind, on standpipes, §86, p26.
Pressures required for hydrants, Table of, §85 p40.
Priority of appropriation, §93, p5.
Profiles for sewer construction, §89, p44.
Prorating, §93, p65.
Pump, Sand, §84, p38.
Pumping engines, §85, p60.
engines, Efficiency of, §85, p60.
for water supply, §85, p55.
from sewer trenches, Cost of, §89, p53.
from wells, §93, p19.
from wells, Best power for, §93, p20.
machinery for water supply, Cost of, §85, pp57 59.
water Cost of, §85 p61.
Pumps, Deep-well, §84, p43.
Efficiency of, §85, p60.
Suction, for deep wells, §84, p41.
Purification, Chemical, of manufacturing wastes, §92, p19.
of sewage, Biological, §92, p24.
of sewage by broad irrigation, §92, p28.
of sewage by chemical precipitation, §92 p19.
of sewage by dilution, §92, p17.
of sewage by oxidation, §92, p17.
of sewage by settlement, §92, 21.
of sewage, General methods of, §92, p16.
of sewage, Summary of methods of, §92, p62.
of water, Chemical, §91, p31.

Q

Quality of water, §90, p1.
Quantity of sewage, Gauging of, §87, p49.
of sewage in separate system, §87, p47.
of water obtainable from a stream, §84, p31.
of water as affected by size of city, §84, p6.
of water as affected by social state, §84, p4.
of water as affected by waste, §84, p6
of water for a water supply, §84, p4.
of water required for fires, §84, p9.
of water used for manufacturing, §84, p5.

R

Radial plan for sewerage systems, §88, p6.
Rain gauge, §84, p16.
Rainfall and its relation to irrigation, §93, p13.
and its relation to sewerage, §87, p15.
Contemporary flow of, §87, p27.
Data relating to, §87, p16.
Formulas for rate of, §87, p17.
in cubic feet per second per acre, §87, p21.
Gauging of, §84, p16.
Rainfall—(Continued)
in its relation to water supply, §84, p16.
Proportion of, reaching sewer, §87, p24.
Quantity of, §84, p18.
Rate of, §87, p15.
Ratio of storm flow to, §87, p28.
Ratio of storm water to, §87, p26.
Records of, §87, p15.
Table of, for four American cities, §84, p71.
Table of maximum rates of, §87, p22.
Talbot's formula for, §87, p18.
Uniformity in maximum rate of, §87, p17.
Variations in, §84, p20.
Rate of filtration, §91, p2.
of flow in London sewers, §87, p28.
of flow into sewers, Rational formula for, §87, p31.
of flow into sewers, Talbot's formula for, §87, p34.
of intermittent filtration, §92, p41.
of rainfall, §87, pp15, 17.
of rainfall, Maximum, Uniformity in, §87, p17.
of water consumption, Average, §87, p48.
Rates of rainfall, Table of maximum, §87, p22.
Ratio of storm flow to rainfall, §87, p28.
of storm water to rainfall, §87, p26.
Raw-water inlets for slow sand filters, §91, p14.
Reclamation fund, §93, p66.
law, §93, p66.
Reconnaissance for storage reservoirs, §86, p2.
Records of rainfall, §87, p15.
Reducer, §85, p29.
Regulators in canal head-works, §93, pp38, 40.
Reinforced-concrete dams, §93, p26.
-concrete sewers, §89, p19.
Removal of iron from ground water, §91, p33
of water from sewer trenches, §89, p37.
Removing lead joints, §85, p25.
Reservoir, Distributing-, System, §85, p2.
Reservoirs, Classes of, §86, p1.
Distributing, §86, pp1, 15.
Distributing, Advantages of, §86, p15.
Distributing, Capacity of, §86, p16.
Distributing, Location of, §86, p15.
Distributing, Pipe connections in, §86, p17
for irrigation, §93, p21.
for springs, §93, p23.
Pure-water, in filtration plants, §91, p5.
Storage, §86, pp1, 2.
Storage, Location of, §86, p2.
Storage, Outlet pipe for, §86, p14.
Storage, Required capacity of, §86, p5.
Resistance of gravel to flow of water, §91, p11.
of sand to flow of water, §91, p10.
Ribs, Stiffening, for standpipes, §86, p42.

Ridge-and-furrow system of broad irrigation, §92, p35.
Rights of owners bordering on streams, §86, p13.
River intakes, §84, p50.
Rivers, Pollution of, §90, p1.
Rivet lines for standpipes, Spacing of, §86 p35.
Riveted pipe, Flow in, §85, p49.
pipe for water supply, §85, p47.
pipe, Laying of, §85, p49.
pipes, Joints for, §85, p48.
Rivets, Distance from center of, to edge of plates, §86, p34.
Rochester, Evaporation in, Table of, §84, p72.
Rock excavation for sewers, Cost of, §89, p53.
Loose-, dams, §93, p27.
Rods, Sucker, of a deep-well pump, §84, p45.
Roofs for elevated tanks, §86, p59.
for standpipes, §86, p48.
Roughness of water-supply pipes, §84, p62.
Rubble masonry, Canals lined with, §93, p37.
Run-off, §84, p37.
-off in its relation to irrigation, §93, p15.
Rye, Raising of, by irrigation, §93, p64.

S

Sand bed for filters, §91, p2.
Filter, Washing of, §91, p22.
filters, Slow, Construction of, §91, p4.
for filters, Effective size of, §91, p8.
for filters, Laying of, §91, p13.
for filters, Quality of, §91, p8.
pump, §84, p38.
Resistance of, to flow of water, §91, p10.
Uniformity coefficient of, §91, p9.
Saprophytic bacteria, §92, p11.
bacteria, Beneficial effect of, §92, p12.
Scouring sluices, §93, p40.
Section, Pipe, §85, p17.
Sediment in irrigation water, §93, p10.
Sedimentation, §90, p25.
Clarification of sewage by, §92, p44.
Separate sewerage system, §87, p2.
system of sewerage, Quantity of sewage in, §87, p47.
system of sewers, Outlets for, §89, p31.
Septic tanks, §92, pp44, 63.
tanks, Advantages of, §92, p45.
tanks, Capacity of, §92, p47.
tanks, Changes of sewage in, §92, p45.
tanks, Closed, §92, p46.
tanks, Details of, §92, p49.
tanks, Open, §92, p46.
tanks, Organic and mineral matter in, §92, p46.
Service periods in irrigation, §93, p7.
Service—(Continued)
pipes, §85, p55.
water meters, §85, p36.
Settling basins, §90, p25.
basins, Cleaning of, §90, p29.
basins, Continuous-flow, §90, p26.
basins, Continuous-flow, Clear-water well for, §90, p30.
basins, Continuous-flow, Velocity in, §90, p28.
basins, Depth of, §90, p29.
basins, Fill-and-draw, §90, pp26, 27.
basins, Location of, §90, p29.
basins, Materials for, §90, p27.
Sewage, Albuminoid ammonia in, §92, p9.
Amount of, required for irrigation, §92, p32.
analyses, Interpretation of, §92, p5.
analyses, Table of, §92, p6.
analysis, §92, p1.
analysis, Bacterial, §92, p12.
analysis, Chemical, §92, p5.
Application of, to the ground in broad irrigation, §92, p34.
Ash in, after combustion, §92, p8.
Biological purification of, §92, p24.
Chlorine in, §92, p9.
Combustible matter in, §92, p3.
Composition of, §92, p1.
discharge and water supply, §87, p48.
Discharge of, into streams and lakes, §92, p16.
disposal by broad irrigation, §92, p28.
Dry solids in, §92, p8.
Duty of, §92, p31.
effluent, Test of, §92, p16.
Feces in, §92, p2.
Fertilizing value of, §92, p32.
Filtration of, §92, pp24, 25.
Filtration of, through coarse filters, §92, p50.
Fish test of, §92, p16.
Free ammonia in, §92, p9.
Gaugings of, §87, p49.
Incubation test of, §92, p16.
Intermittent filtration of, §92, pp25, 39.
irrigation (See *Broad Irrigation*).
Nitrates in, §92, p10.
Organic and inorganic matter in, §92, pp1, 7.
Organic nitrogen in, §92, p9.
Oxidation of, by aeration, §92, p27.
Oxygen consumed by, §92, p9.
pollution of rivers and lakes, §90, p1.
Progressive changes in, §92, p4.
purification, Action of bacteria in, §92, pp24, 26.
purification at Massachusetts experimental station, §92, p24.

Sewage—(Continued)
purification by broad irrigation, §92, p28.
purification by chemical precipitation, §92, p19.
purification by chemical precipitation, Disadvantages of, §92, p62.
purification by dilution, §92, pp17, 62.
purification by oxidation, §92, p17.
purification by sedimentation, §92, p44.
purification, Effect of vegetation on, §92, p25.
purification, General methods of, §92, p16.
-purification plant, General arrangement of, §92, p55.
purification, Precipitation plants for, §92, p20.
purification, Reduction of organic matter in, §92, p10.
purification, Summary methods of, §92, p62.
Quantity of, in separate system, §87, p47.
Settling of, §92, p21.
Straining of, through coke, §92, p57.
Urine in, §92, p2.
Variations in the strength of, §92, p4.
Sewer, Brick-, intersections, §89, p26.
calculations, §88, p9.
calculations, Use of diagrams for, §88, p13.
construction, §89, p1.
construction, Materials used in, §88, p28.
-construction notes, §89, p46.
construction, Profiles for, §89, p44.
construction, Working map for, §89, p44.
crossings, §89, p19.
design, General remarks on, §88, p19.
grades, §87, p11.
house connections, §89, p24.
lines, Curves in, §89, p42.
Locating central line of, §89, pp41, 44.
location, Maps for, §87, p8.
pipe, Cast-iron, §88, p41.
pipe, Cost of, §89, p48.
pipe, Cost of laying, §89, p49.
pipe, Double-strength, §88, p31.
pipe, Joints for, §88, pp31, 34.
pipe, Requisites of, §88, p32.
pipe, Standard, §88, p31.
pipe, Standard forms of, §88, p34.
pipe, Strength of, §88, p31.
pipe, Table of thickness of, §88, p47.
pipe, Terra-cotta (See *Terra-cotta*).
pipe, Thickness of, §88, p31.
pipes, Standard dimensions of, §88, p30.
pipes, Stanford joints for, §88, p35.
records, §89, p40.
surveys, §87, p4; §89, p40.
surveys, Record of, §89, p42.
Sewer—(Continued)
tables, §88, pp15, 42.
Y branches for house connections, §89 p24.
Sewerage calculations, Accuracy of, §88, p16.
systems, §87, p1.
systems, Arrangement of, §88, p1.
systems, Comparison of, §87, p2.
systems, Fan plan for, §88, p3.
systems, Intercepting plan for, §88, p3.
systems, Perpendicular plan for, §88, p1.
systems, Radial plan for, §88, p6.
systems, Zone plan for, §88, p6.
Sewers, Brick, §88, p36.
Brick, Construction of, §89, p16.
Brick, Cost of, §89, p50.
Brick, Cross-section of, §88, p36.
Brick, Estimates for, §89, p50.
Brick, Thickness of, §88, p37
Bridges for, §89, p22.
Cement, §88, p40.
Circular brick, Table of velocities and discharges for, §88, p45.
Circular, Calculations for, §88, p9.
Concrete, Construction of, §89, p17.
Condition producing maximum flow in, §87, p24.
Cross-sections for, §88, p20.
Depth of, §87, p14.
Effect of duration of storm on, §87, p23.
Effect of violent storms on, §87, p20.
Egg-shaped, §88, p20.
Egg-shaped, Formulas for, §88, p20.
Egg-shaped, General form of, §88, p20.
Egg-shaped, Practical dimensions of, §88, p27.
Egg-shaped, Table of velocities and discharges for, §88, p46.
Excavation for, §89, pp33, 37.
Foundations for, §89, p12.
House, §87, p2.
Leveling for, §87, pp5, 6.
Location of, §87, pp8, 11.
Materials used for, §88, p36.
Maximum rate of flow in, §87, p28.
Object of, §87, p1.
on different grades, Connection between, §89, p7.
Pipe, Estimates for, §89, p48.
Pipe, Table of velocities and discharges for, §88, p42.
Reinforced-concrete, §89, p19.
Relative capacity of circular and egg-shaped, §88, p24.
Storm, §87, p2.
Trenching for, §89, p33.
Wooden, §88, p40.
Shallow wells, §84, p33.

Shallow—(Continued)
wells, Construction of, §84, p34.
wells, Yield of, §84, p33.
Sheet piling for sewer trenches, §89, p34.
Sheeting for sewer trenches, Cost of, §89, p53.
Shell, Steel, §86, p27.
Shut-off valves, §85, p33.
Silica standard of turbidity, §90, p3.
Silt in irrigation water, §93, p10.
Siphon flush tanks, §89, p39.
Siphons, Inverted, §89, p19.
Site for a storage-reservoir dam, §86, p12.
Size, Effective, of filter sand, §91, p8.
of water-supply pipes, §85, p8.
Slow sand filters, Beds for, §91, p7.
sand filters, Construction of, §91, p4.
sand filters, Covering of, §91, p18.
sand filters, General description of, §91, p1.
sand filters, Operation of, §91, p21.
sand filters, Outflow from, §91, p15.
sand filters, Raw-water for, §91, p14.
Sludge, Disposal of, in chemical-precipitation plants, §92, p23.
in intermittent filtration, §92, p41.
in various sewage-purification processes, §92, pp60, 61.
Table of daily, for 1,000 persons, §92, p61.
Sluices, Emptying, for canals, §93, p40.
Scouring, §93, p40.
Soap solution, Standard, §90, p12.
test for hardness of water, §90, p12.
Socket, Deep-and-wide, for sewer pipe, §88, p31.
Standard, for sewer pipe, §88, p31.
Sockets, Table of depths of, §88, p47.
Soft water, §90, p11.
Softening of hard water, §91, p34.
of hard water, Continuous flow process for, §91, p36.
of hard water, Intermittent process for, §91, p36.
Soil, Preparation of, for irrigation, §93, p54.
Solution, Standard soap, §90, p12.
Source of water supply, Choice of, §84, p2.
Sources of ground water, §93, p19.
of water supply, §84, p16.
Special castings for water-supply pipe lines, §85, p29.
Specials for water-supply pipe lines, §85, p29.
Spigot, §85, p17.
Spillways for irrigation canals, §93, p40.
Spiral riveted pipe, §85, p50.
Springs as a source of water supply, §84, p46.
Storage reservoirs for, §93, p23.
Sprinkling filters, §92, p58.
irrigation, §93, p54.
Stability of standpipes, §86, p26.
Stakes, Grade, for sewers, §89, p44.
Line, for sewers, §89, p44.
Standard sewer pipe, §88, p31.
Silica, of turbidity, §90, p3.
soap solution, §90, p12.
socket for sewer pipe, §88, p31.
thicknesses and weights of cast-iron water-supply pipe, §85, p45.
Standpipes, §86, p19.
Anchorage for, §86, p36.
Bottom plates of, §86, p35.
Capacity of, §86, p20.
Design and construction of, §86, p27.
Diameter of, §86, p20.
Effect of ice on, §86, p44.
Foundations for, §86, p38.
Height of, §86, p20.
Joints between plates of, §86, p29.
Joints for, §86, p34.
Location of, §86, p19.
Materials for, §86, p27.
Methods of delivery from, §86, p43.
Painting of, §86, p46.
Roofs for, §86, p42.
Stability of, §86, p26.
Stiffening ribs for, §86, p42.
Thickness of, §86, p24.
Thickness of, Table of, §86, p28.
Vertical joints for, §86, p33.
Standard joints for sewer pipes, §88, p35.
Stave pipe, §85, p51.
pipe, Laying of, §85, p55.
Staves, Pipe, Table of dimensions of, §85, p54
Steel, Flange, §80, p28.
pipe for water supply, §85, p47.
pipe for water supply, Laying of, §85, p49.
Shell, §86, p27.
Tank, §86, p27.
Stone, Canals line with, §95, p35.
Storage for water supply, Estimating quantity of, §86, p5.
reservoirs, §86, pp1, 2.
reservoirs for irrigation, §93, p21.
reservoirs for wells, §93, p23.
reservoirs, Location of, §86, p2.
reservoirs, Outlet pipes for, §86, p14.
reservoirs, Required capacity of, §86, p5.
Water, for irrigation, §93, p21.
Water, Table of, §86, p8.
Storm flow, Coefficient of, §87, p29.
sewers, §87, p2.
water, §87, p2.
water, Contemporary flow of, §87, p27.
water, Flow of, on different surfaces, §87, p25.
water, Flow of, per acre, Formulas for, §87, pp32, 36.

Storm—(Continued)
water, Ratio of, to rainfall, §87, p26.
-water sewerage system, §87, p2.
Storms, Effect of duration of, on sewers, §87, p23.
Effect of violent, on sewers, §87, p20.
Stream compensation, §86, p13.
Quantity of water obtainable from, §84, p31.
Streams as sources of water supply, §84, p25.
Discharge of sewage into, §92, p16.
Gauging of, §84, p25.
Gauging of, for irrigation, §93, p15.
Yield of, §84, p27.
Street washings in sewage, §92 p3.
Strength of sewer pipe, §88, p31.
Submerged water-supply pipes, §85, p20.
Subsoil irrigation, §93, p58.
Subsurface irrigation, §92, p59.
Sucker rods of a deep-well pump, §84, p45.
Suction pumps for deep wells, §84, p41.
Sulphate, Ferrous, Removal of, from water, §91, p33.
Supports for elevated tanks, §86, p48.
Surface velocities, §87, pp34, 36.
water for irrigation, §93 p11.
Surveying the site for a reservoir dam, §86, p12.
Surveys for irrigation canals, §93, p28.
for irrigation storage reservoirs, §93, p22.
for sewers, §87, p4.
for storage reservoirs, §86, p2.
for water supply, §85, p5.
Preliminary, for irrigation, §93, p12.
Sewer, §89, p40.
Sewer, Record of, §89, p42.
System, Sewerage, Combined, §87, p2.
Sewerage, Separate, §87, p2.
Sewerage, Storm-water, §87, p2.
Systems, Sewerage, Comparison of, §87, p2.

T

Table of approximate formulas for flow in cast-iron pipe, §84, p74.
of bricks required for brick sewers, §89, p50.
of bricks required for manholes, §89, p54.
of chemicals that will soften permanently hard water, §91, p35.
of chemicals that will soften temporarily hard water, §91, p34.
of coefficients of storm flow, §87, p30.
of copper-sulphate proportions uninjurious to fish, §91, p32.
of cost and efficiency of precipitants for sewage purification, §92, p22.
of cost of pumping from sewer trenches, §89, p53.

Table—(Continued)
of costs of laying sewer pipe, §89, p49.
of costs of various types of engines, §85, p58.
of daily average water consumption for various months, §84, p68.
of daily sludge for 1,000 persons, §92, p61.
of daily water consumption in different cities, §84, p68.
of depths of sockets, §88, p47.
of dimensions of egg-shaped sewers, §88, p47.
of dimensions of pipe staves, §85, p54.
of discharge coefficients for gravel, §91, p11.
of effective velocities for a slope of 1 per cent., §87, p35.
of excreta from one person in 1 year, §92, p2.
of ingredients of mortar, §89, p51.
of maximum rates of rainfall, §87, p22.
of minimum grades for pipe sewers, §87, p12.
of monthly evaporation for Boston and Rochester, §84, p72.
of mortar for brick sewers, §89, p51.
of number of bacteria found at different depths, §92, p30.
of organic and mineral matter in septic tanks, §92, p47.
of pipe sewer laid with one barrel of cement, §89, p49.
of pressures required for hydrants, §85, p40.
of prices of sewer pipe, §89, p48.
of purification of sewage by oxidation, §92, p18.
of rainfall for four American cities, §84, p71.
of ratio of maximum to average water consumption, §84, p69.
of reactions, shears, etc., for elevated-tank circular vertical girders, §86, p51.
of reactions, shears, etc., for elevated-tank horizontal circular girders, §86, p51.
of relative impervious area, §87, p26.
of results of chemical precipitation of sewage, §92, p21.
of sewage analyses, §92, p6.
of sewage effluent from coarse filters, §92, p51.
of sewage purification by chemical methods, §92, p5.
of sewer velocities, §87, p38.
of thicknesses and weights of cast-iron water-supply pipe, §85, p45.
of thicknesses of sewer pipe, §88, p47.
of thicknesses of standpipe plates, §86, p28.
of turbidity of water, §90, p5.
of velocities and discharges for circular brick sewers, §88, p45

Table—(Continued)
of velocities and discharges for egg-shaped sewers, §88, p46.
of velocities and discharges for pipe sewers, §88, p42.
of water consumption according to population, §84, p65.
of water consumption as affected by meters, §84, p66.
of water consumption for various purposes, §84, p66.
of water consumption in Boston and suburbs, §84, p65.
of water required for domestic and fire purposes for different populations, §84, p70.
of water required for fires, §84, p67.
of water storage per square mile of watershed, §86, p9.
of yield of watersheds, §84, p73.
showing loss of organic matter in sewage-purification plant, §92, p10.
Tables, Sewer, §88, pp15, 42.
Talbot's formula for rainfall, §87, p18.
formula for rate of flow into sewers, §87, p34.
Tank steel, §86, p27.
Tanks, Elevated (See *Elevated Tanks*).
Filter, Material for, §91, p27.
Flush, §89, p38.
Septic, §92, pp44, 63.
Upward-filtration §92, p49.
Tastes in water, §90, p9.
in water, Reduction of, by filtration, §91, p31.
Tee connecting pipe, §85, p30.
Temperature, Effect of, on broad irrigation, §92, p34.
Temporarily hard water, Softening of, §91, p34.
Terra-cotta pipe, General description of, §88, p28.
-cotta pipe, Making of, §88, p29.
Test, Fish, of sewage, §92, p16.
Hammer, of water-supply pipe, §85, p14.
Incubation, of sewage, §92, p16.
Soap, for hardness, §90, p12.
Testing water, Methods of, §90, p14.
Tests of sewage effluent, §92, p16.
Physical, of water, §90, p14.
Thickness of bottom of elevated tank, §86, p46.
of brick sewers, §88, p37.
of cast-iron pipe for water supply, §85, p43.
of concrete sewers, §88, p39.
of manhole walls, §89, p4.
of riveted steel pipe, §85, p49.
of sewer pipe, §88, p31.
Thickness—(Continued)
of sewer pipe, Table of, §88, p47.
of standpipes, §86, p24.
of standpipes, Table of, §86, p28.
Tilting flush tanks, §89, p39.
Timber dams, §93, p23.
flumes, §93, p45.
flumes, Design of, §93, p47.
flumes, Flow in, §93, p47.
foundations for sewers, §89, p15.
Towers for elevated tanks, §86, p48.
Transit notes of sewer surveys, §89, p42.
Trenches, Sewer, Cost of back filling, §89, p54.
Sewer, Cost of pumping from, §89, p53.
Trenching for sewers, §89, p33.
Trestles for flumes, §93, p50.
Tubercules, §84, p62.
Tuberculation, §84, p62.
Tunnels for irrigation conduits, §93, p53.
Turbidity, §90, p2.
Measure of, §91, p3.
Permissible, §90, p5.
Table of, §90, p5.
Turnouts for canals, §93, p44

U

Underdrains for slow sand filters, §91, p5.
Uniformity coefficient of sand, §91, p9.
Units used in measuring water for irrigation, §93, p5.
Upward filtration, Advantages and disadvantages of, §92, p63.
-filtration tanks, §92, p49.
Urine in sewage, §92, p2.
Use, Beneficial, Principle of, §93, p65.

V

Valve flush tanks, §89, p39.
Valves, Air, §85, p31.
Check-, §85, p34.
in water-supply systems, §85, p31.
Shut-off, §85, p33.
Vegetation, Effect of, on sewage purification, §92, p23.
Velocities in sewers for various grades, Table of, §87 p38.
Surface, §87, pp34, 36.
Velocity in continuous-flow settling basins, §90, p28.
in sewers as affected by depth of flow, §88, p17.
Limiting, in earthen canals, §93, p32.
Venturi meter, §85, p35.

W

Walls, Manhole, Thickness of, §89, p4.
Washing of filter sand, §91, p22.
Waste of water, §84, p6; §85, p37.

Water, Albuminoid ammonia in, §90, p19.
Amount of, required for crops, §93, p2.
analyses compared with sewage analyses, §92, p6.
analyses, Interpretation of, §90, pp14, 22.
analysis, Bacteriological, §90, p23.
analysis, Chemical, §90, pp17, 22.
analysis, Collecting samples for, §90, p15.
Chemical purification of, §91, p31.
Chlorine in, §90, p18.
Color of, §90, p6.
consumption according to population, Table of, §84, p65.
consumption as affected by meters, Table of, §84, p66.
consumption, Average rate of, §87, p48.
consumption, Factors governing, §84, p4
consumption for various months, Daily average, Table of, §84, p68.
consumption for various purposes, Table of, §84, p66.
consumption, Formulas for, §84, p14.
consumption in Boston and suburbs, Table of, §84, p65.
consumption in different cities, Table of, §84, p68.
consumption, Ratio of maximum to average, Table of, §84, p69.
consumption, Variations in, §84, p10; §87, p48.
Dangerous elements in, §90, p16.
distribution, Methods of, §85, p1.
Examination of, §90, p14.
filtration, §91, p1.
for irrigation, Losses of, §93, pp16, 18.
for irrigation, Quality of, §93, p10.
for irrigation, Quantity of, §93, p11.
for irrigation, Sediment in, §93, p10.
for irrigation, Silt in, §93, p10.
for irrigation, Sources of, §93, p11.
for irrigation, Units used for, §93, p5.
Free ammonia in, §90, p20.
General sanitary examination of, §90, p16.
Ground, Removal of iron from, §91, p33.
Ground, Sources of, §93, p19.
hammer, Allowance for, in water-supply pipe, §85, p44.
Hard, §90, p11.
Hard, Objections to, §90, p13.
Hard, Softening of, §91, p34.
Hardness of, §90, p11.
Impurities in, §90, p1.
Impurities in, compared with those in sewage, §92, p6.
Metallic substances in, §90, p17.
meter, Foote's, §93, p60.
meters, §85, p35.

Water—(Continued)
meters, Advantages of, §84, p7.
meters, Service, §85, p36.
Methods of testing, §90, p14.
Microscopic examination of, §90, p23.
Necessity of, for raising crops, §93, p1.
Nitrates in, §90, p21
Nitrites in, §90, p21.
Odors in, §90, p9.
Organic matter in, §90, p19.
Oxygen-consuming capacity of, §90, p22.
Polluted, as a source of infection, §92, p14.
Quality of, §90, p1.
Quantity of, for water supply, §84, p4.
Quantity of, obtainable from a stream, §84, p31.
Removal of, from sewer trenches, §89, p37.
required for domestic and fire purposes according to population, Table of, §84, p69.
required for fires, §84, p67.
Resistance of gravel to flow of, §91, p11.
Resistance of sand to flow of, §91, p10.
Soft, §90, p11.
storage for irrigation, §93, p21.
-storage table, §86, p8.
Storm, §87, p2.
supply, Advantages of good, §84, p1.
supply and sewage discharge, §87, p48.
supply, Choice of source of, §84, p2.
supply, Design of pipe systems for, §85, pp5, 9.
supply, Maps for, §85, p5.
supply, Quantity of, §84, p4.
-supply pipe, Kinds and properties of, §85, p43.
-supply pipe lines, Construction of, §85, p13.
-supply pipe system, Filling of, §85, p28.
-supply pipes, Leaks in, §85, p26.
-supply pipes, Location and size of, §85, p6.
-supply pipes, Submerged, §85, p20.
supply, Sources of, §84, p16.
supply, Surveys for, §85, p5.
Tastes in, §90, p9.
for irrigation, Duty of, §93, p5.
for irrigation, Measurement of, §93, p59.
Waters, Chalybeate, §90, p10.
Iron-impregnated, §90, p10.
Watershed, §84, p27.
Average flow from, §84, p30.
Minimum flow from, §84, p30.
Study of, for irrigation purposes, §93, p12.
Watersheds, Table of yield of, §84, p73.
Wedges and blocks under water-supply pipe, §85, p19.

Weight of cast-iron pipe for water supply §85, p44.
of cast-iron pipe line, §85, p46.
of lead in pipe joints, §85, p20.
Weir, Diverting, §89, p28.
formula used in irrigation, §93, p15.
Leaping, §89, p29.
measurements of water for irrigation, §93, p60.
Weirs for irrigation head-works, §93, p19.
Wells, Artesian, §84, p33.
as sources of water supply, §84, p32.
Battery of, §84, p37.
Classes of, §84, p33.
Deep, §84, p33.
Deep, Advantages and disadvantages of, §84, p45.
Deep, Driving of, §84, p37
Deep, Machines for drilling, §84, p37.
Deep, Methods of delivery from, §84, p41.
Deep, Operation of, §84, p46.
Deep, Pumps for, §84, p43.
Deep, Suction pumps for, §84, p41.
Deep, Yield of, §84, p36.
Dug, §84, p33.
Pumping from, §93, p19.
Shallow, §84, p33.
Wells—(Continued)
Storage reservoirs for, §93, p23.
Wheat, Raising of by irrigation, §93, p64.
Wind bracing for elevated tanks, §86, p57.
pressure on elevated tanks, §86, p57.
pressure on standpipes, §86, p26.
Windmills, §93, p20.
Cost of, §93, p21.
Wooden sewers, §88, p40.
-stave pipe, §85, p51.
-stave pipe, Laying of, §85, p55.
Worcester, Precipitation plant at, §92, p23.
Working map for sewer construction, §89, p44.
Works, Head-, of canals, §93, p38.
Wye connecting pipe, §85, p30.

Y

Y's, §85, p30.
Yarner, §85, p17.
Yield of a deep well, §84, p36.
of a shallow well, §84, p33.
of a stream, §84, p27.
of watersheds Table of, §84, p73.

Z

Zone plan for sewerage systems, §88, p6.
Zymotic diseases, §92, p11.